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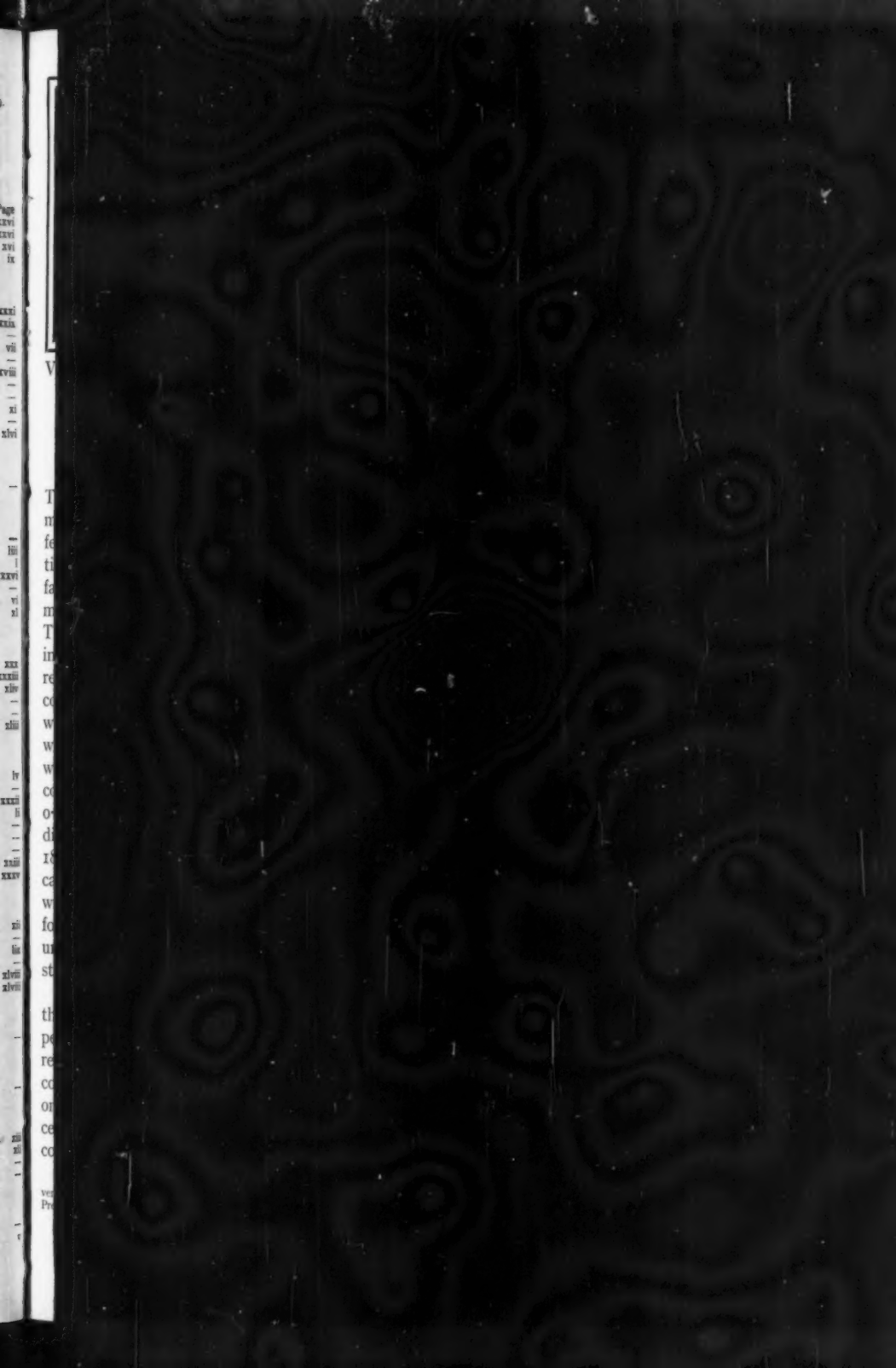

CONCRETE
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Volume LIV, No. 2. LONDON, FEBRUARY, 1959.

EDITORIAL NOTES

The Effect of Calcium Chloride on Reinforcement.

THE possibility that the use of calcium chloride to hasten the hardening of concrete may lead to corrosion of steel reinforcement has been known for many years, but few tests seem to have been made on the subject in this country until the Institution of Civil Engineers arranged for an investigation to be made following a failure in Canada in the year 1953 in a prestressed concrete pipeline thirty-seven miles long, which was thought to be due to corrosion of the prestressing wires. This question is of much greater importance in the case of prestressed concrete, in which wires as thin as one-twelfth of an inch in diameter are used, than in reinforced concrete containing much larger principal reinforcement, for a depth of corrosion that would not seriously reduce the cross-sectional area of a large bar would penetrate completely through a thin wire. Reports on these tests, which were undertaken at Leeds University, are now available.* Some 344 specimens were made and tested in the laboratory, and prestressed pipes made under factory conditions were also examined. The steel used included wires from 0·128 in. to 0·276 in. diameter and mild steel and bright steel bars of $\frac{1}{4}$ in., $\frac{3}{8}$ in., and $\frac{1}{2}$ in. diameter, and many of the specimens were cured in steam at a temperature of 180 deg. Fahr. Corrosion of the steel occurred in all the specimens containing calcium chloride and cured by this means, whereas in the steam-cured specimens without calcium chloride there was no corrosion. This result alone is not a reason for banning the use of calcium chloride in concrete containing steel, for it seems unnecessary to add a chemical to hasten the hardening of concrete if expensive steam-curing plant be installed for the same purpose.

It was found, however, that patches of rust and discoloration occurred on the steel in many specimens of concrete with crushing strengths up to 8000 lb. per square inch and cured in air. Great variations were noticeable in the occurrence of corrosion. In some specimens the steel was bright and free from discoloration after twenty-six months, whereas in others corrosion occurred after only five weeks in specimens made with 1 : $1\frac{1}{2}$: 3 concrete and containing 2 per cent. of calcium chloride by weight of the cement. In the case of specimens containing calcium chloride and cured for eight hours in water at a temperature

* "Use of Calcium Chloride in Prestressed Concrete." By R. H. Evans, Professor of Civil Engineering, Leeds University. Proceedings of the World Conference on Prestressed Concrete, San Francisco, 1957. Also "Applications of Prestressed Concrete to Water Supply and Drainage", by R. H. Evans, Proc. Inst. C.E., Vol. 4, Part III, 1955.

of 180 deg. Fahr. there was much less corrosion than in specimens cured in steam; the author expresses the opinion that a longer period in hot water might have resulted in corrosion occurring in all cases, but again it is not likely that the chemical would be used in concrete cured in hot water. It seems that corrosion of the steel will certainly occur if concrete containing calcium chloride is subjected to steam curing at the temperature used in these tests, and that corrosion may occur if such concrete is allowed to harden naturally, even although the cover is dense high-strength concrete $1\frac{7}{8}$ in. thick and whether or not it is made with Portland cement, Portland blastfurnace cement, or sulphate-resisting cement as in these tests. The author recommends the use of steam curing without calcium chloride, but it is obvious that this method of curing should never be used if calcium chloride is included in the mixture.

The experiments proved that the corrosion was electrolytic, and varied with the electrical potential. It was found that the addition of calcium chloride and curing in wet steam generally increased the potential about three times, and in one case by ten times, compared with concrete without calcium chloride. When the specimens were not subjected to steam until up to six months after casting it was found that the older the concrete the less the corrosion ; except in the case of structures that will be subjected to steam this is of academic interest only, for the only purpose of steam curing is to obtain high strength as soon as possible after casting. Calcium chloride is hygroscopic, and it was found to be difficult to dry concrete containing this chemical.

In the author's opinion the hard drawn steel commonly used in prestressed concrete in this country is not subject to stress corrosion. Also, it has not been proved that the heat-treated steel used on the Continent is particularly susceptible to stress corrosion, with the possible exception of one type of high-tensile wire made in Germany by a hot-rolling and tempering process. In the author's experiments the amount of stress in the wire was not related to the amount of corrosion of the steel in the specimens containing calcium chloride.

In a discussion of the results, the author points out that the alkalinity of concrete is a powerful preventive of corrosion, and refers to experiments made in Russia in mixing concrete with lime-water instead of plain water to increase the alkalinity of the concrete. It was found that no corrosion occurred in concrete containing up to 7 per cent. of calcium chloride when calcium hydroxide was added to the mixing water in the proportion of 5 grammes per litre, although 2 per cent. of calcium chloride caused corrosion when plain water was used. It is stated by investigators in the U.S.S.R. that sodium silicate and sodium benzoate added to the mixing water also counteract the corrosive effects of calcium chloride.

The latest report published in this country on other tests on the use of calcium chloride in reinforced concrete seems to be a note in the Report of the Building Research Board for the year 1951, which states that tests on 1 : 6 concrete containing calcium chloride showed that there was slight corrosion after two years' exposure. It is not to be expected that the amount of corrosion in specimens stored in a dry atmosphere will increase at later ages, but the tests at Leeds University indicate that if concrete containing steel is later subjected to very damp conditions the potential immediately rises and conditions are produced that favour corrosion starting again in a hitherto unreactive concrete. More long-period research is needed on this subject.

A Doubly-curved Shell Roof in Belgrade.

Design and Construction of Exhibition Hall.

By Dipl.-Ing. M. KRSTIĆ (Belgrade).

THREE large pavilions and several smaller structures have been built at the new fair ground in Belgrade. One of the main buildings, namely the pavilion for heavy industrial products, is described in this article.

The requirements for the hall were: (1) An area of 43,000 sq. ft. for the exhibition of heavy machinery at ground level; (2) A gallery of 10,800 sq. ft. for lightweight exhibits; (3) Basement storage facilities under the whole area of the hall; (4) A single-span roof in order to allow the use of the pavilion for sporting and public events; (5) The entire structure had to be of reinforced concrete.

A hall (*Fig.* 1) with a doubly-curved shell roof was decided upon. The

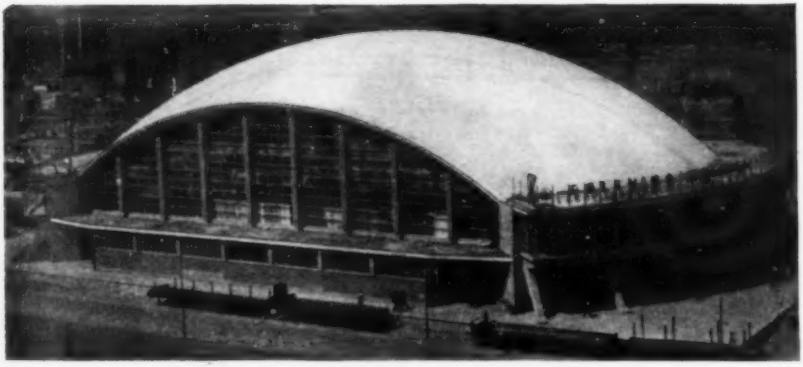


Fig. 1.—General View of Hall.

shape of the roof is defined by the surface formed by the intersection of one horizontal and two parallel vertical planes disposed symmetrically about the centre of a sphere about 320 ft. in diameter. The lengths of the major and minor axes of the horizontal projection of the surface are 230 ft. and 157 ft. (*Fig.* 2), and the normal thickness of the shell is $3\frac{1}{2}$ in. increasing gradually from a distance of 11 ft. 6 in. from the edges to a maximum of $7\frac{7}{8}$ in. The shell is stiffened by edge-beams to form a statically-stable system. The edge-beams on the longer side are supported by vertical columns and on the shorter side by inclined columns positioned radially as shown in *Figs.* 3 and 5.

At the corners, where the tensile stresses in the concrete exceed 57 lb. per square inch, the roof is prestressed with high-tensile steel cables, each of which comprises six $\frac{3}{16}$-in. wires with an ultimate tensile strength of 114·3 tons per square inch. These cables were positioned in the approximate direction of the lines of principal tensile stress (*Fig.* 4). In the portions of the roof which are entirely in compression, and where the tensile stresses are less than 57 lb. per square inch, a network of $\frac{1}{4}$-in. mild steel bars at about 10 in. centres is used

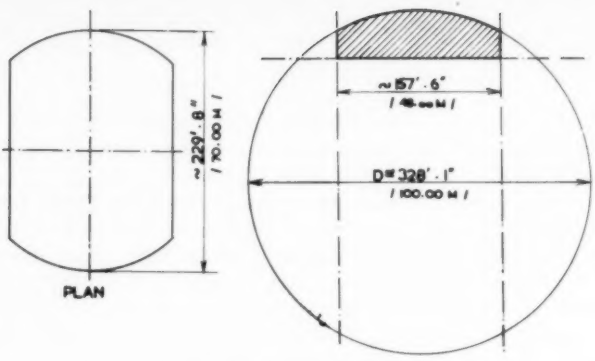


Fig. 2.—Shape of Hall.

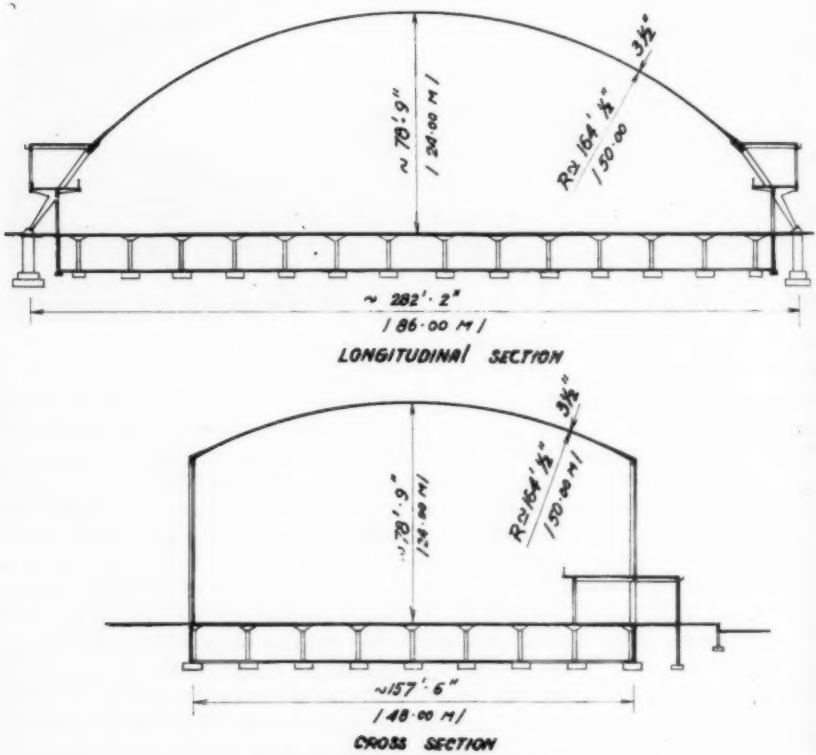


Fig. 3.—Sections through Hall.

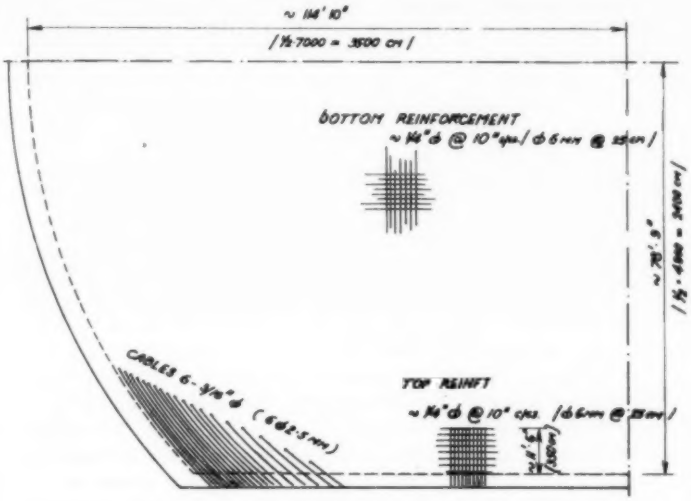


Fig. 4.—Reinforcement of Roof.

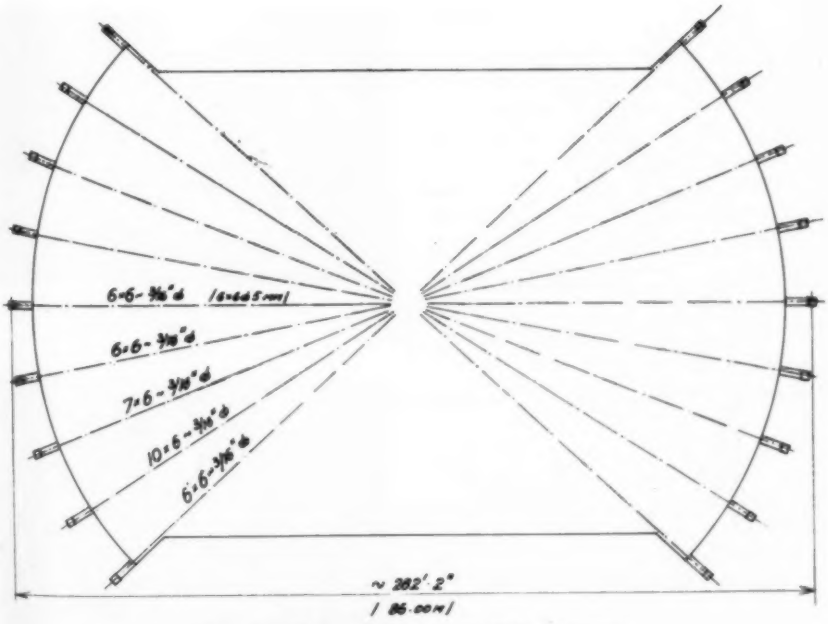


Fig. 5.—Ties restraining Radial Columns.

as bottom reinforcement throughout. In the thickened edges, a similar mesh is provided at the top. The edge-beams are partially prestressed, cables comprising six high-tensile wires of $\frac{3}{16}$ in. diameter being used for this purpose.

The ground on which the hall is constructed offers little resistance to horizontal forces; the foundations of the inclined columns are therefore connected by ties capable of resisting the horizontal components of the inclined thrusts of the shell. These ties pass through the ground-floor slab and each of them joins two radially-opposed columns (*Fig.* 5). High-tensile steel cables were used, and, in order to prevent movements due to the elongation of the ties, all the cables were tensioned before the scaffolding was removed.

The galleries were designed as slabs supported by beams, and flat-slab construction was used for the ground floor. The columns are supported on separate foundations of reinforced and plain concrete. Tests indicated that considerable settlements could be expected, especially at the inclined columns, and to allow for uneven settlement provision is made for raising the inclined columns in the future. For this purpose the blocks supporting the inclined columns are separated from the foundations, and spaces for two hydraulic jacks are provided in the upper part of each foundation (*Fig.* 6). All the inclined columns were raised by $\frac{3}{16}$ in. before lowering the centering to the roof, thus preventing initial settlement as well as that which could be expected as a result of uneven lowering of the centering (*Fig.* 7).

The shuttering for the roof (*Figs.* 8 and 9) consisted of timber panels about

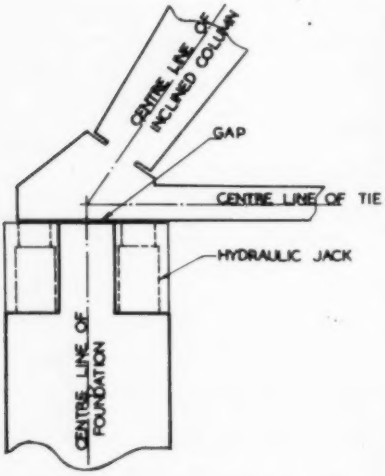


Fig. 6.—Arrangement of Jacks.

Fig. 7.—Radial Column after Jacking.

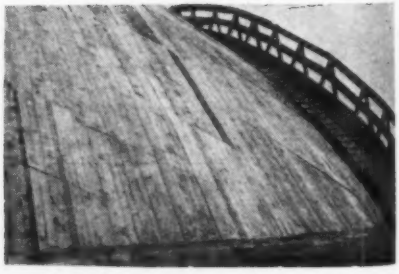


Fig. 8.—View of Shuttering.

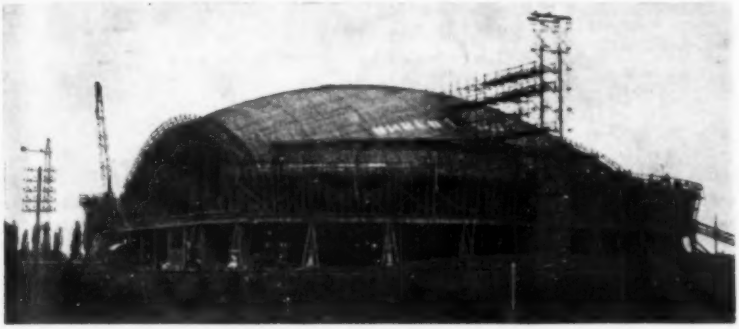


Fig. 9.—Shuttering for Roof.

10 ft. by 5 ft. which were also used for two similar roofs over an adjacent building. The shuttering was supported by tubular steel scaffolding comprising props of 2 in. diameter at about 10 ft. centres and horizontal ties at about 6 ft. 9 in. centres.

The excavation for the basement and foundations was commenced in November 1956 and all the concreting was completed on 15 May 1957, despite difficulties due to frost and snow.

Design of Roof.

The shell was designed in accordance with the membrane theory, assuming a symmetrical vertical load. No theory is available for the analysis of an unsymmetrically-loaded shell of such a shape, but as the live loads do not exceed 20 per cent. of the total load, and the stresses in the shell are less than 285 lb. per square inch (20 kg. per square centimetre) this condition was ignored.

Considering the equilibrium of an element of the shell in the co-ordinate system (δ, β) (*Fig.* 10), the following system of partial differential equations was derived for the unknown forces N_δ, N_β, and $N_{\delta\beta}$.

$$N_\delta + N_\beta = -ag\cos\delta\cos\beta \quad . \quad . \quad . \quad . \quad (1a)$$

$$\frac{\partial}{\partial\delta}(N_\delta\cos\delta) + N_\beta\sin\delta + \frac{\partial N_{\delta\beta}}{\partial\beta} = -ag\sin\delta\cos\delta\cos\beta \quad . \quad (1b)$$

$$\frac{\partial N_\beta}{\partial\beta} - N_{\delta\beta}\sin\delta + \frac{\partial}{\partial\delta}(N_{\delta\beta}\cos\delta) = -ag\cos\delta\sin\beta \quad . \quad . \quad (1c)$$

This system leads to

$$\frac{\partial^2 N_\delta}{\partial\delta^2} - 5\frac{\partial N_\delta}{\partial\delta}\tan\delta - 2N_\delta(1 - 2\sin^2\delta) + \frac{1}{\cos^2\delta}\cdot\frac{\partial^2 N_\delta}{\partial\beta^2} = 2ag\cos\delta\cos\beta \quad (2)$$

By inserting $N_\delta = F(\delta)\cos\beta$ in equation (2) the following ordinary differential equation is obtained.

$$\frac{d^2F}{d\delta^2} - 5\frac{dF}{d\delta}\tan\delta - 3F(1 - \tan^2\delta) = 2ag\cdot\frac{1}{\cos\delta}.$$

The solution of this equation is

$$F(\delta) = C_1 \frac{1}{\cos^3 \delta} + C_2 \frac{\sin \delta}{\cos^3 \delta} - ag \frac{1}{\cos \delta}, \text{ or}$$

$$N_\delta = \left(C_1 \frac{1}{\cos^3 \delta} + C_2 \frac{\sin \delta}{\cos^3 \delta} - ag \frac{1}{\cos \delta} \right) \cos \beta \quad . \quad . \quad (3)$$

It is postulated that $N_\delta = 0$ at the edges of the shell, since the edge-beams cannot resist forces normal to the plane of the beams. Hence the boundary conditions for the determination of the constants C_1 and C_2 are $N_\delta = 0$ when $\delta = \delta_0$ and $\delta = -\delta_0$. Substituting these in equation (3), $C_1 = ag \cos^2 \delta_0$ and $C_2 = 0$. Hence

$$N_\delta = \frac{ag}{\cos^3 \delta} (\cos^2 \delta_0 - \cos^2 \delta) \cos \beta \quad . \quad . \quad . \quad . \quad (4)$$

$$N_\beta = ag \left[\frac{1}{\cos^3 \delta} (\cos^2 \delta_0 - \cos^2 \delta) - \cos \delta \right] \cos \beta \quad . \quad . \quad (5)$$

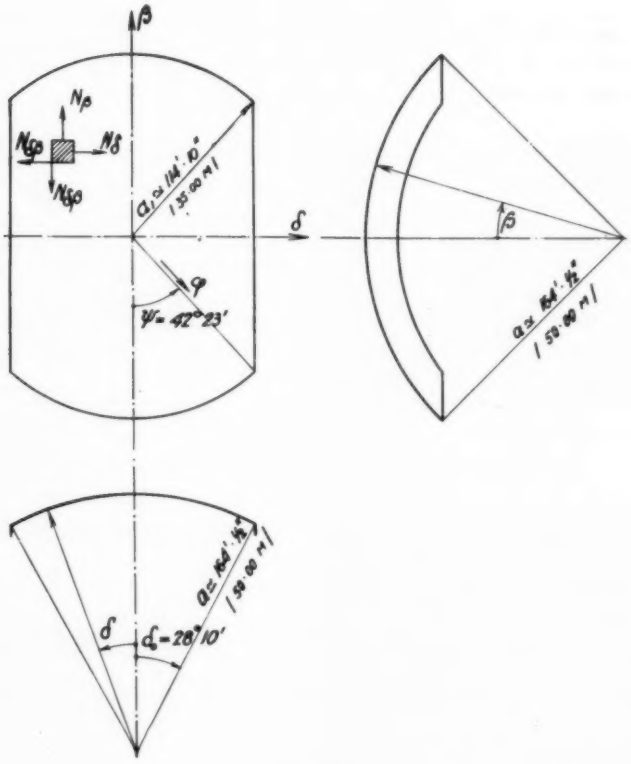


Fig. 10.

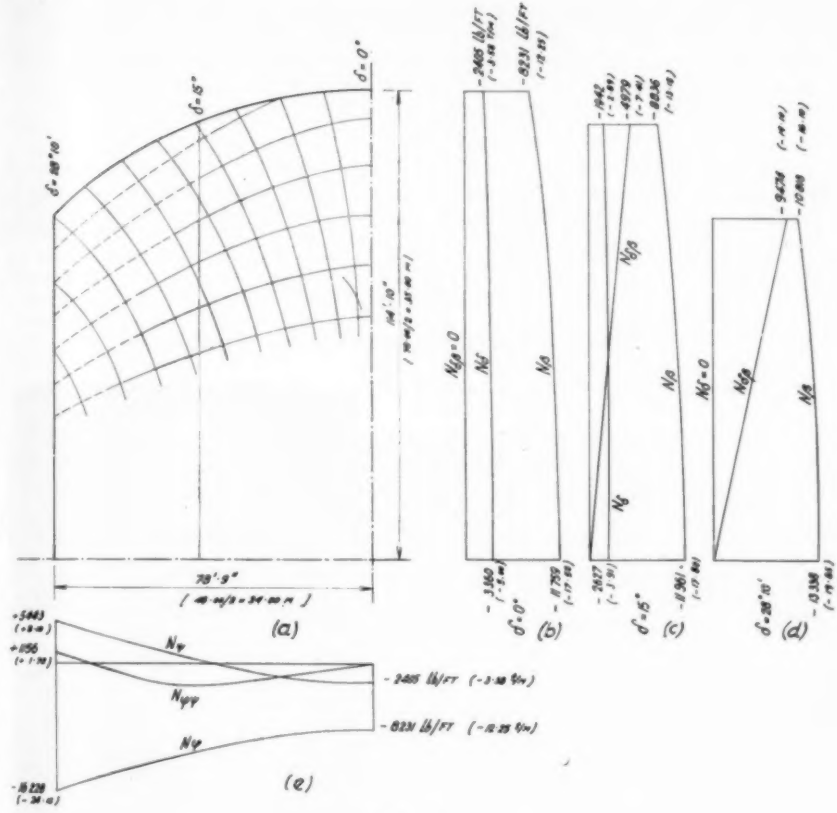


Fig. 11.

$$N_{\delta\beta} = -\,ag\frac{\sin\delta}{\cos^3\delta}(\cos^2\delta_0 + \cos^2\delta)\sin\beta \quad . \quad . \quad . \quad (6)$$

Diagrams of the forces N_δ, N_β, and $N_{\delta\beta}$ for the sections corresponding to $\delta = 0$, $\delta = 15°$ and $\delta = 28° 10'$ are shown in *Fig.* 11, together with the main trajectories of stress projected on a horizontal plane.

Bending moments occur at the ends of the shell, due mainly to the uneven stresses in the edges of the shell and the edge-beams. This problem was not solved analytically. By comparison with a circular dome it was estimated that these moments cannot exceed 29,500 in.-lb. and that they occur only in a narrow zone adjacent to the edge-beams.

The edge-beams on the longer side of the hall were calculated as continuous beams of nine spans, loaded with their own weight and the tangential forces $N_{\delta\beta}$ (*Fig.* 11*d*). The axial tensile force is resisted by prestressing cables, and the bending moments and shearing forces are resisted by mild-steel reinforcement. The edge-beams at the shorter side of the hall are continuous over eight

spans. They are designed to withstand the vertical load due to their own weight and the roofs over the galleries, and the radial and tangential forces N_ϕ and $N_{\phi\psi}$ respectively ; the stresses were therefore converted into a spherical co-ordinate system (ψ, ϕ) for the shorter edge of the shell. *Fig.* 11*e* shows graphically the forces N_ϕ and $N_{\phi\psi}$. The axial force due to $N_{\phi\psi}$ in the beam is resisted by prestressing cables and the remaining forces by mild steel reinforcement.

The architect was Mr. M. Pantović, and the consulting engineer for the foundation work was Dr.-Ing. R. Vucetić. The " Rad " concern of Belgrade carried out the building work under the guidance of Dipl.-Ing. D. Taušanović, the chief site engineer.

The Late Professor H. N. Walsh.

It is with much regret that we record the death of Professor H. N. Walsh, which occurred suddenly on November 27 last.

Professor Walsh was born in 1892. He was appointed to the chair of civil engineering at the University of Cork in 1921 before he was thirty years of age, and retired in 1957 at the age of 65. For many years he was a member of the Senate of the University, and was Dean of the Engineering Faculty. From 1940 to 1942 he was President of the Institution of Civil Engineers of Ireland, from 1938 to 1940 he was a member of the Dáil Eireann Drainage Commission, and he was a member of the Commission for Industrial Research and Standards since its inception. He was concerned with the large Shannon hydro-electric works carried out in the 1930's, and had previously (in 1920) published, with P. O. Donaill, an important work on the possibilities of the development of hydro-electric power in Ireland.

Walsh had a remarkable flair for teaching and for gaining the affection of his students. He combined academic distinction with much practical experience, and his constant endeavour was to persuade his students that engineering sense was more important than arithmetic. He was always an enthusiast for concrete, but had little use for much of the work that is done to-day in the name of research. One of his major interests was the making of good concrete, and his book " How to Make Good Concrete " is a model of a practical approach to this very practical problem, and it is typical of the man that in this work and elsewhere he discards the water–cement ratio method of expressing the relative proportions of cement and water in concrete in favour of the much simpler cement–water ratio.

His two hobbies were angling and photography, at both of which he excelled; he had occasionally published articles on angling, and on his retirement he started to write a book on the subject. Walsh will be missed by an exceptionally large number of friends who enjoyed his company or who sought his advice on matters relating to concrete.

A Prestressed Sea-wall.

PRESTRESSED precast concrete units with post-tensioned steel are being used to repair the Flaghead sea-wall at Poole, Dorset. The wall was built about sixty years ago, and was strengthened in 1933; the present work, which was started in November 1957, is necessary due to the deterioration of the surface.

As the face of the wall is at an angle of 60 deg. to the horizontal, precast units were considered to be preferable to concrete cast in place under tidal conditions. A cross section is shown in *Fig.* 2. The prestressing is intended to ensure that the facing will be rigid and airtight, and is designed to allow the replacement of any parts which may be damaged or become unserviceable. The prestress is applied by means of the Lee-McCall system, the advantages of which in this case are stated to include the small number of prestressing operations, the ease with which the bars may be threaded through the ducts and the anchors assembled, the ease with which damaged units may be replaced, and the smaller surface area of steel that might be exposed to corrosion in a sea-wall; the surface area of a 1¼-in. bar is 42 sq. in. per foot, compared with 104 sq. in. per foot in the case of an equivalent number of 0·276-in. wires.

Expansion joints are provided at intervals of 40 ft. The precast facing beams are about 39 ft. long and 2 ft. 3 in. deep, and the coping units about 10 ft. long; the four coping units in each bay are prestressed together after they are placed in position, and the facing beams are prestressed at the factory.

A longitudinal and transverse prestress of 500 lb. per square inch is provided by means of two longitudinal 1-in. bars in each facing beam, two longitudinal 1¼-in. bars in the coping, and groups of three transverse 1¼-in. bars at 5 ft. centres. The initial tensioning stress in each bar is 42 tons per square inch, and losses are estimated to be about 15 per cent. Light mild-steel reinforcement is provided at each anchorage.

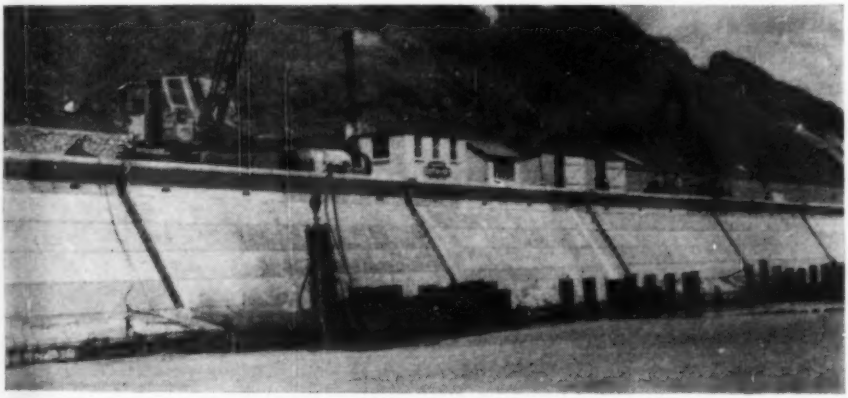


Fig. 1.—Prestressed Facing Slabs.

The precast units are made in a factory. The concrete is made with 3 parts of gravel aggregate of ¾ in. maximum size and 1 part rapid-hardening Portland cement. The specified minimum cube strengths are 6500 lb. per square inch in compression at 28 days and 5250 lb. per square inch when the prestress is applied; the average strength obtained at seven days is 6370 lb. per square inch, and at 28 days 8250 lb. per square inch. The moulds for the units comprise concrete beds and side shutters of timber faced with steel. The longitudinal ducts are formed by means of rubber tubes stiffened with removable steel rods, and the transverse ducts with timber cores in the form of folding wedges.

Three days after casting a prestress of 250 lb. per square inch is applied to the facing beams, which are then lifted from the casting beds and stacked in an upright position. The full prestress is applied seven days after casting, both bars in each unit being tensioned simultaneously, and the longitudinal ducts are filled with colloidal cement grout. The coping units are not prestressed before they are delivered.

Construction.

The sequence of construction is as follows. Light steel sheet piling, 15 ft. long, with a copper content of between 0·25 and 0·35 per cent., is driven to form a temporary cofferdam the top of which is 4 ft. above Ordnance Datum. The existing concrete apron and steel sheet piling are demolished to the level of the underside of the new base-block and concrete pro-

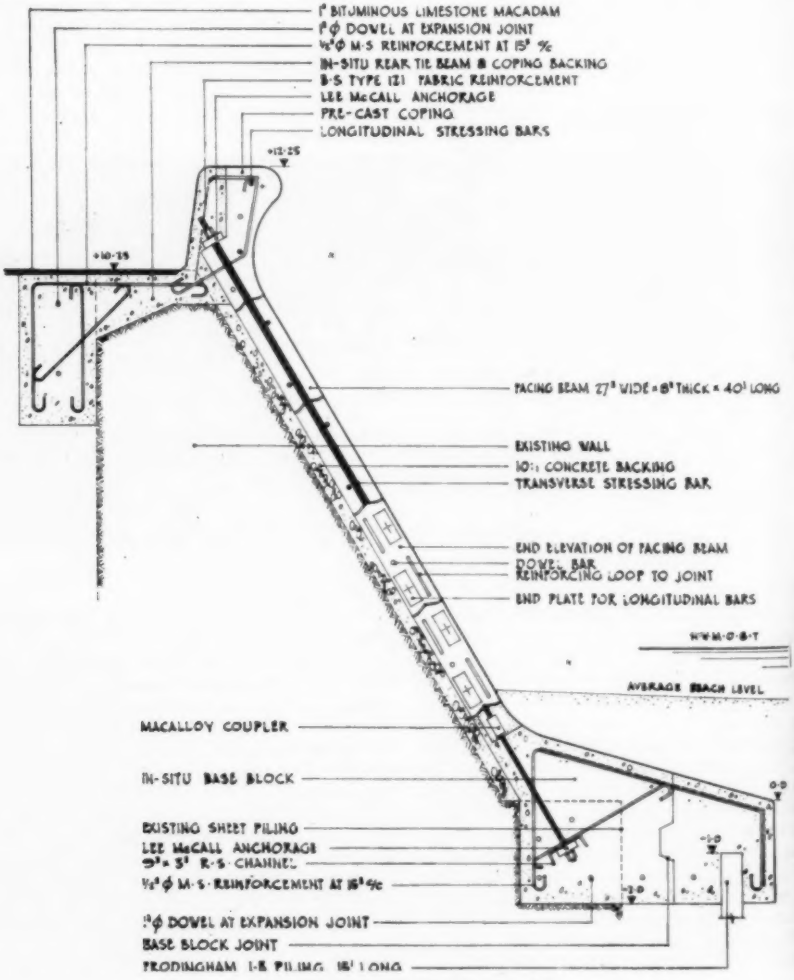


Fig. 2.—Typical Cross Section.

files are placed, one behind each expansion joint and two in each bay. The bottom facing-beam is placed in position, and serves to position and align the transverse prestressing bars and to serve as a shutter for the upper face of the base-block on which the facing-beam eventually rests. The transverse prestressing bars are then placed in position and supported by means of tubular scaffolding; the bottom anchors bear on a steel channel which is supported on small concrete piers. Pockets are left around the couplers; the shuttering and reinforcement for the landward part of the base-block are prepared, placed in position on a previously-prepared base, levelled, and strutted. Concrete with an aggregate-cement ratio of 6 : 1 is placed up to the level of the curved portion of the block; the curved shutter is then placed in position, and concrete is placed behind the facing-beam until it is about 1 ft. higher than the top of the base-block and begins to flow from holes formed for the purpose near the top of the shutter.

The upper lengths of the prestressing bars are removed and the rest of the facing-beams and the coping units are placed in position; each unit is bedded on cement mortar ¼ in. thick. The prestressing bars are then replaced, the joints between the coping units concreted, the anchor-plates bedded on cement mortar, and nuts and washers are placed on the

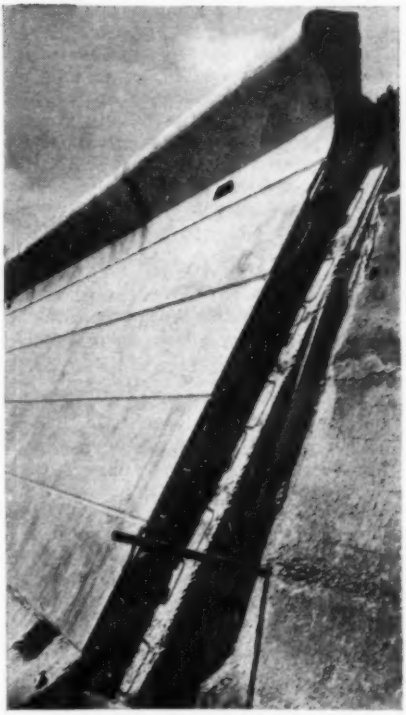


Fig. 3.—Detail of Joint.

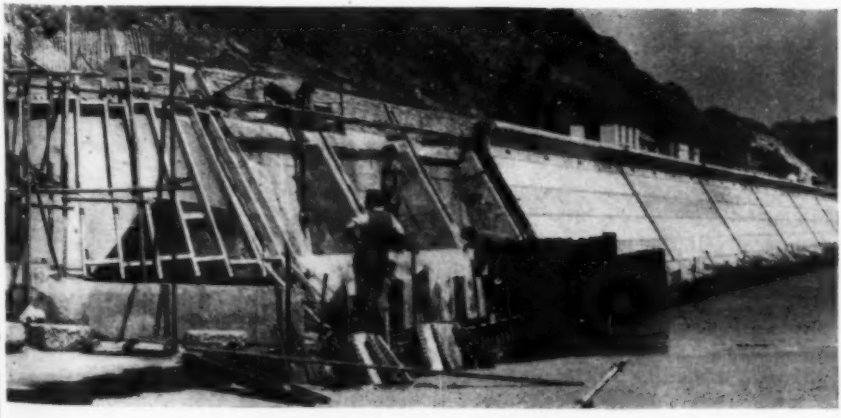


Fig. 4.—Stages in Construction.

prestressing bars, which are tensioned three days later. The longitudinal bars in the copings are tensioned first, the elongation being 1·84 in. at 42 tons per square inch; the transverse bars have an elongation of about 0·463 in., varying slightly according to the extent of the bond between the lower part of the bar and the base-block. The steel sheet piling is then re-driven to the level shown in *Fig.* 2 and the seaward section of the base-block is cast.

Spaces about 12 in. wide and 8 in. deep are provided between the ends of the facing-beams for the formation of the expansion joints. The reinforcement consists of mild steel loops projecting from the ends of the beams, and 1-in. dowel-bars between the beams which are anchored to the existing wall by $\frac{5}{8}$-in. hooked bars. The jointing material is $\frac{1}{2}$ in. wide, and a rubber water-stop 6 in. wide is provided. The joints are concreted in lifts of 2 ft. 3 in.; the aggregate-cement ratio is 4·5 and the maximum size of the aggregate is $\frac{3}{8}$ in.

A ground beam is cast behind the existing wall, and the coping is completed with a layer of concrete cast in place to protect the anchors of the prestressing bars. A concrete ramp and steps are formed at one end of the wall, and a return wall at the other.

The costs of the work per linear foot of wall, based on the tender submitted in August 1957, are as follows. Temporary works, £6; site preparation, 8*s.*; excavation, £1 2*s.* 7*d.*; piling 15 ft. long, including lifting, pitching, and driving 14 ft., £10 15*s.* 6*d.*; base-block, £3 19*s.* 4*d.*; facing-beams, £2 1*s.* 6*d.*; coping, £3 15*s.*; transverse prestressing, including rolled steel channel, £9 12*s.* 6*d.*; profiles and concrete behind the new work, £1 19*s.*; expansion joints, £1 2*s.* 1*d.*; rear beam and backing to coping, £2 13*s.* 6*d.*; surfacing to promenade, 16*s.* The total cost per linear foot of wall is £44 5*s.*; the contract price for the work is £28,614.

The design and construction of the work are under the supervision of the Borough Engineer, Mr. J. R. Barron, A.M.I.C.E., in collaboration with McCalls Macalloy, Ltd. The general contractors are Messrs. Blackford & Son (Calne), Ltd., and the precast concrete is made at Abingdon, Berks, by the Cowley Concrete Co., Ltd.

"Concrete and Constructional Engineering" Prize Design.

THE price of £25, awarded annually by the proprietors of this journal for competition amongst the students at the Department of Concrete Technology, Imperial College of Science and Technology, London, has been awarded to Mr. A. W. S. Wojakowski for the year 1957–1958. The subject was set by Professor A. L. L. Baker, and was for a design for a factory and office building in precast concrete. The assessor was Mr. Leslie Turner, M.I.C.E., a former President of the Institution of Structural Engineers.

The Late F. J. Samuely.

MR. F. J. SAMUELY, who practised in London as a consulting engineer, died on January 22 at the age of 57. Mr. Samuely was born in Vienna, and worked in Vienna and Berlin before he came to London about twenty-five years ago. In recent years he was prominent in developing the use of precast structural members and prestressed concrete, and collaborated with architects in some notable reinforced and prestressed buildings at the Brussels Exhibition and the Festival of Britain in London in 1951. He acted as consultant for many important buildings, including the United States Embassy now in course of construction in London, and many schools and tall residential flats in which he made much use of precast members.

Application of the Plastic-Hinge Method to a Multiple-Story Frame.

Frame with Three Unequal Bays.

By E. W. TOKARSKI, A.M.I.C.E., D.I.C.

THE exact elastic analysis of a building frame such as that shown in *Fig.* 1 is impracticable, and approximations must be made to simplify the problem. Such an assumption is that the points of contraflexure in columns occur mid-way between the floors; this ignores the exact requirements of the elastic theory and can be justified only when considering the plastic effects which occur just before failure. The theory of plastic hinges[1] offers a practical method of analysing building frames which may be highly indeterminate, and this article shows the application of the theory to the design of a multiple-story frame with three unequal bays. The procedure to be followed is summarised in the following.

(1) Select plastic hinges at the positions of the maximum bending moments, obtained by the elastic theory, due to the loading required. Bending moments can be superimposed only if they produce the same set of plastic hinges.

(2) Derive the expressions for the rotation of hinges in terms of the unknown plastic-hinge restraints.

(3) Assume values for the plastic-hinge moments which give the most economical distribution of bending moments.

(4) Check that the rotations are within safe limits, and, if they are not, adjust the assumed values of plastic-hinge moments.

Positions and Rotations of Plastic Hinges.

Positions of plastic hinges for the frame shown in *Fig.* 1 have been chosen for the combination of vertical load and wind load acting from the left. The members of the framework, which is assumed to have been made statically determinate by the development of sufficient plastic hinges, are deformed elastically between the hinges by the action of applied bending moments M_o and m_o and plastic-hinge restraining moments $\bar{X}$ acting at each hinge. The resulting rotations of the plastic hinges, that is the differences in the slopes of the ends of the members adjacent to the hinges, are obtained from the general expressions I, which are

$$\text{I} \quad \begin{bmatrix} \delta_{11} & \delta_{12} & \cdots & \delta_{1(n-1)} & \delta_{1n} \\ \delta_{21} & \delta_{22} & \cdots & \delta_{2(n-1)} & \delta_{2n} \\ \vdots & \vdots & \vdots & \vdots & \vdots \\ \delta_{n1} & \delta_{n2} & \cdots & \delta_{n(n-1)} & \delta_{nn} \end{bmatrix} \begin{bmatrix} \bar{X}_1 \\ \bar{X}_2 \\ \vdots \\ \bar{X}_n \end{bmatrix} + \begin{bmatrix} \delta^\circ_1 \\ \delta^\circ_2 \\ \vdots \\ \delta^\circ_n \end{bmatrix} = -\begin{bmatrix} e_1 \\ e_2 \\ \vdots \\ e_n \end{bmatrix}$$

the well-known elastic equations adapted to suit the ultimate-load condition. In these, δ_{ik} is the influence coefficient denoting the rotation of hinge i due to a unit moment at hinge k in the direction of $\bar{X}_k$, $\left(\text{which equals } \int_L \frac{m_i m_k}{EI} d_s\right)$; δ°_i

(*Continued on page 88.*)

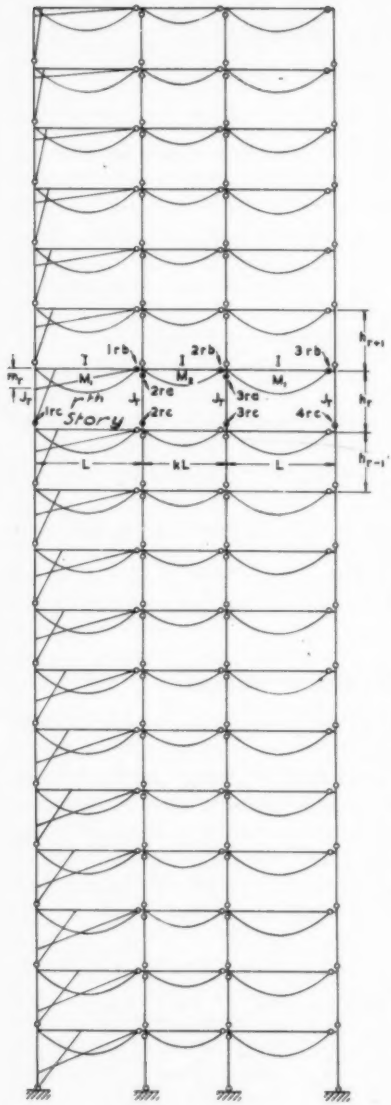


Fig. 1.—Assumed Positions of Hinges and Free Bending Moments for Wind and Vertical Loads.

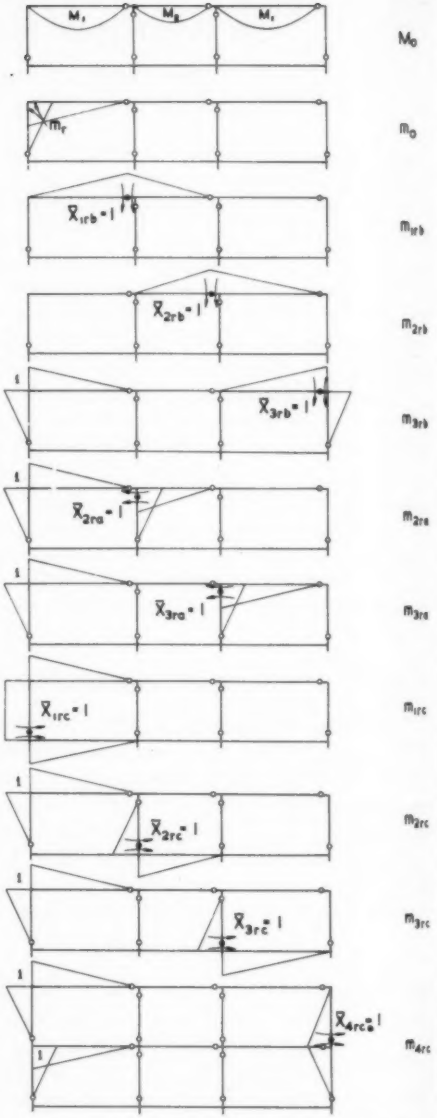


Fig. 2.—Influence Diagrams for Hinges of the r^{th} Story.

II

$$-\theta_{1rb} = \frac{L}{EI}\left[-\frac{M_1}{3} - \frac{kM_2}{3} - \frac{m_r}{6} + \frac{1+k}{3}\bar{X}_{1rb} + \frac{k}{6}\bar{X}_{2rb} + \frac{1}{6}\bar{X}_{3rb} - \frac{2k-1}{6}\bar{X}_{2ra} + \frac{1}{6}\bar{X}_{3ra}\right.$$
$$\left. - \frac{a-1}{6}\bar{X}_{1rc} - \frac{2ka-1}{6}\bar{X}_{2rc} + \frac{1}{6}\bar{X}_{3rc} - \frac{a-1}{6}\bar{X}_{4rc}\right] \quad 2.1$$

$$-\theta_{2rb} = \frac{L}{EI}\left[-\frac{M_1}{3} - \frac{kM_2}{3} + \frac{k}{6}\bar{X}_{1rb} + \frac{1+k}{3}\bar{X}_{2rb} + \frac{1}{6}\bar{X}_{3rb} - \frac{k}{6}\bar{X}_{2ra} - \frac{1}{6}\bar{X}_{3ra}\right.$$
$$\left. - \frac{ka}{6}\bar{X}_{2rc} - \frac{a}{6}\bar{X}_{3rc}\right] \quad 2.2$$

$$-\theta_{3rb} = \frac{L}{EI}\left[-\frac{2M_1}{3} - \frac{1+y}{3}m_r + \frac{1}{6}\bar{X}_{1rb} + \frac{1}{6}\bar{X}_{2rb} + \frac{2(1+y)}{3}\bar{X}_{3rb} + \frac{1+y}{6}\bar{X}_{2ra}\right.$$
$$+ \frac{1+2y}{6}\bar{X}_{3ra} - \frac{2a-2-3y}{6}\bar{X}_{1rc} + \frac{1+y}{3}\bar{X}_{2rc} - \frac{a-2-2y}{6}\bar{X}_{3rc}$$
$$\left. - \frac{2a+4ay-y-2}{6}\bar{X}_{4rc}\right] \quad 2.3$$

$$-\theta_{3ra} = \frac{L}{EI}\left[-\frac{M_1}{3} + \frac{kM_2}{3} - \frac{1+y}{3}m_r - \frac{2k-1}{6}\bar{X}_{1rb} - \frac{k}{6}\bar{X}_{2rb} + \frac{1+y}{6}\bar{X}_{3rb}\right.$$
$$+ \frac{1+k+2y}{3}\bar{X}_{2ra} + \frac{1+y}{3}\bar{X}_{3ra} - \frac{2a-2-3y}{6}\bar{X}_{1rc} + \frac{2ka+2+y}{6}\bar{X}_{2rc}$$
$$\left. + \frac{1+y}{3}\bar{X}_{3rc} - \frac{(a-1)(1+y)}{3}\bar{X}_{4rc}\right] \quad 2.4$$

$$-\theta_{3ra} = \frac{L}{EI}\left[-\frac{1+y}{3}m_r + \frac{1}{6}\bar{X}_{1rb} - \frac{1}{3}\bar{X}_{2rb} + \frac{1+2y}{6}\bar{X}_{3rb} + \frac{2+y}{3}\bar{X}_{2ra} + \frac{2(1+y)}{3}\bar{X}_{3ra}\right.$$
$$\left. - \frac{2a-2-3y}{6}\bar{X}_{1rc} + \frac{1+y}{3}\bar{X}_{2rc} + \frac{2+2a+y}{6}\bar{X}_{3rc} - \frac{(a-1)(1+y)}{3}\bar{X}_{4rc}\right] \quad 2.5$$

$$-\theta_{1rc} = \frac{L}{EI}\left[\frac{2(b-1)-3y}{6}m_r - \frac{b-1}{6}\bar{X}_{1rb} - \frac{2(b-1)-3y}{6}\left(\bar{X}_{2rb} + \bar{X}_{2ra} + \bar{X}_{3ra}\right)\right.$$
$$- \frac{a+b-2-3y}{3}\bar{X}_{1rc} - \frac{2(b-1)-3y}{6}\left(\bar{X}_{2rc} + \bar{X}_{3rc}\right)$$
$$\left. - \frac{2(b-1)+(a-1)(2+3y)}{6}\bar{X}_{4rc}\right] \quad 2.6$$

$$-\theta_{2rc} = \frac{L}{EI}\left[-\frac{M_1}{3} + \frac{kM_2}{3} - \frac{1+y}{3}m_r - \frac{2kb-1}{6}\bar{X}_{1rb} - \frac{kb}{6}\bar{X}_{2rb} + \frac{1+y}{3}\bar{X}_{3rb}\right.$$
$$+ \frac{2(1+kb)+y}{6}\bar{X}_{2ra} + \frac{1+y}{3}\bar{X}_{3ra} - \frac{2(a-1)-3y}{6}\bar{X}_{1rc}$$
$$\left. + \frac{1+k+2y}{3}\bar{X}_{2rc} + \frac{1+y}{3}\bar{X}_{3rc} - \frac{(a-1)(1+y)}{3}\bar{X}_{4rc}\right] \quad 2.7$$

$$-\theta_{3rc} = \frac{L}{EI}\left[-\frac{1+y}{3}m_r + \frac{1}{6}\bar{X}_{1rb} - \frac{b}{3}\bar{X}_{2rb} - \frac{b-2(1+y)}{6}\bar{X}_{3rb} + \frac{1+y}{3}\bar{X}_{2ra}\right.$$
$$+ \frac{2(1+b)+y}{6}\bar{X}_{3ra} - \frac{2(a-1)-3y}{6}\bar{X}_{1rc} + \frac{1+y}{3}\bar{X}_{2rc}$$
$$\left. + \frac{2(1+y)}{3}\bar{X}_{3rc} - \frac{(a-1)(1+y)}{3}\bar{X}_{4rc}\right] \quad 2.8$$

$$-\theta_{4rc} = \frac{L}{EI}\left[\frac{(b-1)+y(sb-1)}{3}m_r - \frac{b-1}{6}\bar{X}_{1rb} - \frac{2(b-1)+y(4sb-1)}{6}\bar{X}_{3rb}\right.$$
$$- \frac{(b-1)+y(sb-1)}{3}\left(\bar{X}_{2ra} + \bar{X}_{3ra}\right) - \frac{2(a+b-2)+3y(sb-1)}{6}\bar{X}_{1rc}$$
$$\left. - \frac{(b-1)+y(sb-1)}{3}\left(\bar{X}_{2rc} + \bar{X}_{3rc}\right) - \frac{2(a+b-2)+y[a+sb-4(1+s)]}{6}\bar{X}_{4rc}\right] \quad 2.9$$

For hinges in the top story, $a = 0$; for hinges in the bottom story, $b = s = 0$.

is the rotation of hinge i caused by the external loads $\left(\text{which equals } \int_L \frac{M_o m_i}{EI} d_s + \int_L \frac{m_o m_i}{EI} d_s\right)$; and $\bar{X}_i$ is the assumed value of the plastic-hinge moment at hinge i.

Influence diagrams for hinges of a typical r^{th} story are given in *Fig.* 2, in which the diagrams are plotted on the sides of the members which are in tension. Rotation of any particular hinge in the r^{th} story can be affected by the applied moments and by the restraints at some of the hinges in the story considered, as well as those in the adjacent stories. To obtain expressions suitable for graphical representation, certain assumptions are made and limiting values are introduced for a wide range of cases, as follows.

(1) Both end spans of length L ; shorter inner span of length kL.

(2) I constant for all beams.

(3) J constant for all columns in the same story.

Therefore $y = \frac{Ih_r}{JL}$. Also $\frac{h_{r-1}}{J_{r-1}} = \frac{sh_r}{J_r}$; the limits of s are 0·66 and 1.

(4) Vertical loading on all beams may be either uniformly distributed or concentrated at the third-points (since both give the same value of δ°_i in terms of the maximum ordinate of the free bending-moment diagram). The maximum ordinate of the free bending-moment diagram in both of the end spans is M_1, and in the inner spans is M_2.

(5) Values of m and $\bar{X}$ in the $(r+1)^{th}$ story with regard to the terms containing m are a times the values for the r^{th} story ; the limits of a are 0·75 and 1·0.

(6) Values of m and $\bar{X}$ in the $(r-1)^{th}$ story with regard to terms containing m are b times the values for the r^{th} story ; the limits of b are 1·25 and 1·0.

With these assumptions, equations II for the rotation of plastic hinges can be obtained.

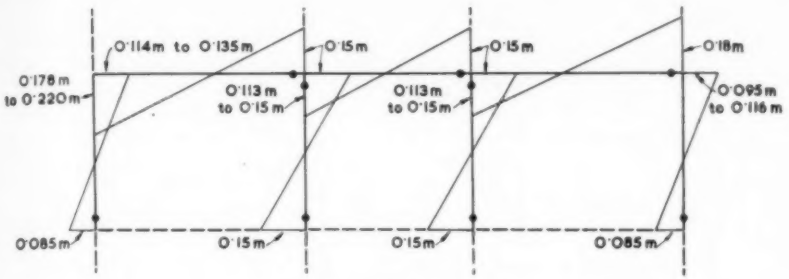
Values of Plastic-Hinge Moments.

A full redistribution of moments in the ultimate plastic condition occurs when the moments at the support and at mid-span caused by the vertical loading, and the moments at both ends of the columns caused by the sway moments, become equal. The achievement of this ideal condition may not be possible, and the designer should try to obtain the most economical distribution of moments that is compatible with the formation of plastic hinges at the assumed sections without the development of excessive rotations.

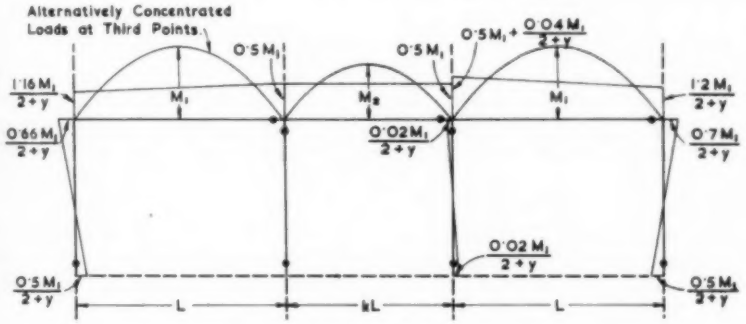
The plastic-hinge moments in equations III will be satisfactory for many practical cases and can always be used as the first approximation. These values

$$\text{III} \quad \left\{ \begin{array}{ll} \bar{X}_{1rb} = \bar{X}_{2rb} = 0{\cdot}15m + 0{\cdot}5M_1 & \bar{X}_{3rb} = 0{\cdot}18m + \frac{1{\cdot}2M_1}{2+y} \\ \bar{X}_{2ra} = \bar{X}_{2rc} = 0{\cdot}15m & \bar{X}_{3ra} = \bar{X}_{3rc} = 0{\cdot}15m - \frac{0{\cdot}02M_1}{2+y} \\ \bar{X}_{1rc} = 0{\cdot}085m - \frac{0{\cdot}5M_1}{2+y} & \bar{X}_{4rc} = 0{\cdot}085m + \frac{0{\cdot}5M_1}{2+y} \end{array} \right\} \quad 3$$

result in the distribution of bending moments shown in *Fig.* 3, in which the requirements of statics are satisfied as the moments at each joint and the shearing forces in the story are balanced.



Due to Wind Load.



Due to Vertical Load.

Fig. 3.—Resultant Maximum Bending Moments on Members of a Typical Story.

IV

$$\theta_{1rb} = \frac{M_1 L}{EI}\left[\frac{0{\cdot}167y - 0{\cdot}25ky - 0{\cdot}5k + 0{\cdot}14}{2+y} + 0{\cdot}333kr_M + (0{\cdot}025k - 0{\cdot}013)r_m\right] \quad 4.1$$

$$\theta_{2rb} = \frac{M_1 L}{EI}\left[\frac{0{\cdot}167y - 0{\cdot}25ky - 0{\cdot}5k + 0{\cdot}12}{2+y} + 0{\cdot}333kr_M + (0{\cdot}02 - 0{\cdot}025k)\,r_m\right] \quad 4.2$$

$$\theta_{3rb} = \frac{M_1 L}{EI}\left[\frac{0{\cdot}213y + 0{\cdot}206}{2+y} + (0{\cdot}013y + 0{\cdot}013)r_m\right] \quad 4.3$$

$$\theta_{2ra} = \frac{M_1 L}{EI}\left[\frac{0{\cdot}113y + 0{\cdot}25ky + 0{\cdot}5k + 0{\cdot}113}{2+y} - 0{\cdot}333kr_M + (0{\cdot}006y - 0{\cdot}025k + 0{\cdot}048)r_m\right] \quad 4.4$$

$$\theta_{3ra} = \frac{M_1 L}{EI}\left[\frac{-0{\cdot}05y - 0{\cdot}007}{2+y} + (0{\cdot}006y + 0{\cdot}028)\,r_m\right] \quad 4.5$$

$$\theta_{1rc} = \frac{M_1 L}{EI}\left[\frac{-0{\cdot}08y}{2+y} + (0{\cdot}025y + 0{\cdot}002)\,r_m\right] \quad 4.6$$

$$\theta_{2rc} = \frac{M_1 L}{EI}\left[\frac{0{\cdot}113y + 0{\cdot}25ky + 0{\cdot}5k + 0{\cdot}113}{2+y} - 0{\cdot}333kr_M + (0{\cdot}006y - 0{\cdot}019k + 0{\cdot}048)\,r_m\right] \quad 4.7$$

$$\theta_{3rc} = \frac{M_1 L}{EI}\left[\frac{-0{\cdot}05y - 0{\cdot}007}{2+y} + (0{\cdot}006y + 0{\cdot}036)\,r_m\right] \quad 4.8$$

$$\theta_{4rc} = \frac{M_1 L}{EI}\left[\frac{0{\cdot}1y}{2+y} + (0{\cdot}016y + 0{\cdot}002)\,r_m\right] \quad 4.9$$

TABLE I.—Influence Coefficients for Y_m in Equations (5) and (8).

Influence coefficient of ↓ on →	$\bar{X}_{1rb}$	$\bar{X}_{2rb}$	$\bar{X}_{3rb}$	$\bar{X}_{2ro}$	$\bar{X}_{3ro}$	$\bar{X}_{1rc}$	$\bar{X}_{2rc}$	$\bar{X}_{3rc}$	$\bar{X}_{4rc}$
e_{1rb}	$-\frac{1+k}{3}$	$-\frac{k}{6}$	$-\frac{1}{6}$	$\frac{2k-1}{6}$	$-\frac{1}{6}$	$\frac{a-1}{6}$	$\frac{2ka-1}{6}$	$-\frac{1}{6}$	$\frac{a-1}{6}$
e_{2rb}	$-\frac{k}{6}$	$-\frac{k+1}{3}$	$-\frac{1}{6}$	$\frac{k}{6}$	$\frac{1}{3}$	—	$\frac{ka}{6}$	$\frac{a}{3}$	—
e_{3rb}	$-\frac{1}{6}$	$-\frac{1}{6}$	$-\frac{2(1+y)}{3}$	$-\frac{1+y}{3}$	$-\frac{1+2y}{6}$	$\frac{2a-2-3y}{6}$	$-\frac{1+y}{3}$	$\frac{a-2-2y}{6}$	$\frac{2a+4ay-y-2}{6}$
e_{2ro}	$\frac{2k-1}{6}$	$\frac{k}{6}$	$-\frac{1+y}{3}$	$\frac{1+k+2y}{3}$	$-\frac{1+y}{3}$	$\frac{2a-2-3y}{6}$	$-\frac{2ka+2+y}{6}$	$-\frac{1+y}{3}$	$\frac{(a-1)(1+y)}{3}$
e_{3ro}	$-\frac{1}{6}$	$\frac{1}{3}$	$-\frac{1+2y}{6}$	$-\frac{1+y}{3}$	$-\frac{2(1+y)}{3}$	$\frac{2a-2-3y}{6}$	$-\frac{1+y}{3}$	$\frac{2+2a+y}{6}$	$\frac{(a-1)(1+y)}{3}$
e_{1rc}	$\frac{b-1}{6}$	—	$\frac{2(b-1)-3y}{6}$	$\frac{2(b-1)-3y}{6}$	$\frac{2(b-1)-3y}{6}$	$\frac{a+b-2-3y}{3}$	$\frac{2(b-1)-3y}{6}$	$\frac{2(b-1)-3y}{6}$	$\frac{1}{6}\{2(b-1)+(a-1)(2+3y)\}$
e_{2rc}	$\frac{2kb-1}{6}$	$\frac{kb}{6}$	$-\frac{1+y}{3}$	$\frac{2(1+kb)+y}{6}$	$-\frac{1+y}{3}$	$\frac{2(a-1)-3y}{6}$	$-\frac{1+k+2y}{3}$	$-\frac{1+y}{3}$	$\frac{(a-1)(1+y)}{3}$
e_{3rc}	$-\frac{1}{6}$	$\frac{b}{3}$	$\frac{b-2(1+y)}{6}$	$-\frac{1+y}{3}$	$-\frac{2(1+b)+y}{6}$	$\frac{2(a-1)-3y}{6}$	$-\frac{1+y}{3}$	$-\frac{2(1+y)}{3}$	$\frac{(a-1)(1+y)}{3}$
e_{4rc}	$\frac{b-1}{6}$	—	$\frac{1}{6}\{2(b-1)+y(4b-1)\}$	$\frac{b-1+y(4b-1)}{3}$	$\frac{b-1+y(4b-1)}{3}$	$\frac{1}{6}\{2(a+b-2)+5y(ba-1)\}$	$\frac{b-1+y(5b-1)}{3}$	$\frac{b-1+y(5b-1)}{3}$	$\frac{1}{6}\{2(a+b-2)+y(5b+a-4(1+s)\}$

The Design Charts.

Substituting for $\bar{X}$ in equations 2·1 to 2·9, and applying in each case the appropriate limiting value for the maximum rotation, expressions IV are obtained. These can be expressed in the more general form

$$\theta = \frac{M_1 L}{EI}[Y_{M1} + r_M Y_{M2} + r_m Y_m] \quad . \quad . \quad . \quad (5)$$

in which $r_M = \dfrac{M_2}{M_1}$ and $r_m = \dfrac{m_r}{M_1}$

A set of design charts can be prepared in which the values of Y_{M1}, Y_{M2}, and Y_m for each hinge are plotted against y for various values of the parameter k. With the aid of these curves the rotations of the hinges can be quickly determined by using equation (5).

The assumed plastic-hinge moments are satisfactory if the resulting rotations are within the safe limiting values and are positive, except in the case of hinges at which the final moment acts in the direction opposite to that indicated on the influence diagrams. Such changes of sign of both θ and $\bar{X}$ may often occur at some column hinges when the moment m_r due to the wind is small in relation to the moment M_1 due to the vertical load.

When a negative rotation of a hinge is accompanied by a value of $\bar{X}$ which acts in the direction of the unit moment on the influence diagram, a hinge will not form. This may occur in a member which is slender in relation to the adjacent members, and the formation of a hinge in such a member would result in excessive rotation of the stiffer member. It is then best to assume that the slender member remains elastic, and to adjust the values of plastic moments to give zero rotation at the particular hinge. If excessive rotation occurs, it can be reduced by increasing the value of the plastic-hinge moment at the hinge concerned, and any adjustments that are necessary can be made quickly with the aid of the influence coefficients in *Tables* I and II.

Examples.—The following examples illustrate the type of curves that may be obtained for use in the design of frames.

EXAMPLE 1.—Frame with three unequal bays, $k = 0{\cdot}7$. Loading: uniformly distributed or concentrated loads at third points (or both); but generally $r_M = \dfrac{M_2}{M_1} \neq k^2$. Values of plastic-hinge moments to be as shown in *Fig.* 3.

Substituting for k, equations 4.1 to 4.9 reduce to equations V; values of Y_{M1}, Y_{M2}, and Y_m are plotted in *Fig.* 4.

EXAMPLE 2.—Values of plastic-hinge moments as previously, but $k = 0{\cdot}8$ and all beams loaded with uniformly-distributed loading, or concentrated loads at third-points, so that $r_M = k^2$. Substituting for k and r_M, equations 4.1 to 4.9 reduce to equations VI. Expressing these in more general form,

$$\theta = \frac{M_1 L}{EI}[Y_M + r_m Y_m] \quad . \quad . \quad . \quad . \quad (8)$$

Values of Y_M and Y_m are plotted in *Fig.* 5.

TABLE II.—Influence Coefficients for Y_{M_1} and Y_M in Equations (5) and (8).

Influence of → on ↓	$\bar{X}_{1rb}$	$\bar{X}_{2rb}$	$\bar{X}_{3rb}$	$\bar{X}_{2ra}$	$\bar{X}_{3ra}$	$\bar{X}_{1rc}$	$\bar{X}_{2rc}$	$\bar{X}_{3rc}$	$\bar{X}_{4rc}$
Θ_{1rb}	$-\frac{1+k}{3}$	$-\frac{k}{6}$	$-\frac{1}{6}$	$\frac{2k-1}{6}$	$-\frac{1}{6}$	–	$\frac{2k-1}{6}$	$-\frac{1}{6}$	–
Θ_{2rb}	$-\frac{k}{6}$	$-\frac{k+1}{3}$	$-\frac{1}{6}$	$\frac{k}{6}$	$\frac{1}{3}$	–	$\frac{k}{6}$	$\frac{1}{3}$	–
Θ_{3rb}	$-\frac{1}{6}$	$-\frac{1}{6}$	$-\frac{2(1+y)}{3}$	$-\frac{1+y}{3}$	$-\frac{2y+1}{6}$	$-\frac{y}{2}$	$-\frac{1+y}{3}$	$-\frac{1+2y}{6}$	$\frac{y}{2}$
Θ_{2ra}	$\frac{2k-1}{6}$	$\frac{k}{6}$	$-\frac{1+y}{3}$	$-\frac{1+k+2y}{3}$	$-\frac{1+y}{3}$	$-\frac{y}{2}$	$-\frac{2k+2+y}{6}$	$-\frac{1+y}{3}$	–
Θ_{3ra}	$-\frac{1}{6}$	$\frac{1}{3}$	$-\frac{1+2y}{6}$	$-\frac{1+y}{3}$	$-\frac{2(1+y)}{3}$	$-\frac{y}{2}$	$-\frac{1+y}{3}$	$-\frac{4+y}{6}$	–
Θ_{1rc}	–	–	$-\frac{y}{2}$	$-\frac{y}{2}$	$-\frac{y}{2}$	$-y$	$-\frac{y}{2}$	$-\frac{y}{2}$	–
Θ_{2rc}	$\frac{2k-1}{6}$	$\frac{k}{6}$	$-\frac{1+y}{3}$	$-\frac{2k+2+y}{6}$	$-\frac{1+y}{3}$	$-\frac{y}{2}$	$-\frac{1+k+2y}{3}$	$-\frac{1+y}{3}$	–
Θ_{3rc}	$-\frac{1}{6}$	$\frac{1}{3}$	$-\frac{1+2y}{6}$	$-\frac{1+y}{3}$	$-\frac{4+y}{6}$	$-\frac{y}{2}$	$-\frac{1+y}{3}$	$-\frac{2(1+y)}{3}$	–
Θ_{4rc}	–	–	$\frac{(4s-1)y}{6}$	$\frac{(s-1)y}{3}$	$\frac{(s-1)y}{3}$	$\frac{(s-1)y}{2}$	$\frac{(s-1)y}{3}$	$\frac{(s-1)y}{3}$	$-\frac{(1+s)y}{2}$

V

$$\theta_{1rb} = \frac{M_1 L}{ET}\left[\frac{-0{\cdot}008y - 0{\cdot}21}{2+y} + 0{\cdot}233\, r_M + 0{\cdot}005\, r_m\right] \quad 6.1$$

$$\theta_{2rb} = \frac{M_1 L}{ET}\left[\frac{-0{\cdot}008y - 0{\cdot}23}{2+y} + 0{\cdot}233\, r_M + 0{\cdot}003\, r_m\right] \quad 6.2$$

$$\theta_{3rb} = \frac{M_1 L}{ET}\left[\frac{0{\cdot}213y + 0{\cdot}206}{2+y} + (0{\cdot}013y + 0{\cdot}013)\, r_m\right] \quad 6.3$$

$$\theta_{2ra} = \frac{M_1 L}{ET}\left[\frac{0{\cdot}288y + 0{\cdot}463}{2+y} - 0{\cdot}233\, r_M + (0{\cdot}006y + 0{\cdot}03)\, r_m\right] \quad 6.4$$

$$\theta_{3ra} = \frac{M_1 L}{ET}\left[\frac{-0{\cdot}05y - 0{\cdot}007}{2+y} + (0{\cdot}006y + 0{\cdot}028)\, r_m\right] \quad 6.5$$

$$\theta_{1rc} = \frac{M_1 L}{ET}\left[\frac{-0{\cdot}08y}{2+y} + (0{\cdot}025y + 0{\cdot}002)\, r_m\right] \quad 6.6$$

$$\theta_{2rc} = \frac{M_1 L}{ET}\left[\frac{0{\cdot}288y + 0{\cdot}463}{2+y} - 0{\cdot}233\, r_M + (0{\cdot}006y + 0{\cdot}035)\, r_m\right] \quad 6.7$$

$$\theta_{3rc} = \frac{M_1 L}{ET}\left[\frac{-0{\cdot}05y - 0{\cdot}007}{2+y} + (0{\cdot}006y + 0{\cdot}036)\, r_m\right] \quad 6.8$$

$$\theta_{4rc} = \frac{M_1 L}{ET}\left[\frac{0{\cdot}1y}{2+y} + (0{\cdot}016y + 0{\cdot}002)\, r_m\right] \quad 6.9$$

VI

$$\theta_{1rb} = \frac{M_1 L}{ET}\left[\frac{0{\cdot}137y + 0{\cdot}081}{2+y} + 0{\cdot}007\, r_m\right] \quad 7.1$$

$$\theta_{2rb} = \frac{M_1 L}{ET}\left[\frac{0{\cdot}137y + 0{\cdot}061}{2+y}\right] \quad 7.2$$

$$\theta_{3rb} = \frac{M_1 L}{ET}\left[\frac{0{\cdot}213y + 0{\cdot}206}{2+y} + (0{\cdot}013y + 0{\cdot}013)\, r_m\right] \quad 7.3$$

$$\theta_{2ra} = \frac{M_1 L}{ET}\left[\frac{0{\cdot}143y + 0{\cdot}172}{2+y} + (0{\cdot}006y + 0{\cdot}028)\, r_m\right] \quad 7.4$$

$$\theta_{3ra} = \frac{M_1 L}{ET}\left[\frac{-0{\cdot}05y - 0{\cdot}007}{2+y} + (0{\cdot}006y + 0{\cdot}028)\, r_m\right] \quad 7.5$$

$$\theta_{1rc} = \frac{M_1 L}{ET}\left[\frac{-0{\cdot}08y}{2+y} + (0{\cdot}025y + 0{\cdot}002)\, r_m\right] \quad 7.6$$

$$\theta_{2rc} = \frac{M_1 L}{ET}\left[\frac{0{\cdot}143y + 0{\cdot}172}{2+y} + (0{\cdot}006y + 0{\cdot}033)\, r_m\right] \quad 7.7$$

$$\theta_{3rc} = \frac{M_1 L}{ET}\left[\frac{-0{\cdot}05y - 0{\cdot}007}{2+y} + (0{\cdot}006y + 0{\cdot}036)\, r_m\right] \quad 7.8$$

$$\theta_{4rc} = \frac{M_1 L}{ET}\left[\frac{0{\cdot}1y}{2+y} + (0{\cdot}016y + 0{\cdot}002)\, r_m\right] \quad 7.9$$

The values of the plastic-hinge moments have been chosen to give reasonable results for many values of k; the best results, however, will be obtained when k varies between 0·7 and 0·85. For values of k approaching 0·5 or unity it may become necessary to increase or decrease the plastic-hinge moments on the inside columns.

ACKNOWLEDGMENT.—The writer is grateful to Professor A. L. L. Baker, D.Sc., for encouragement and advice in the preparation of this article.

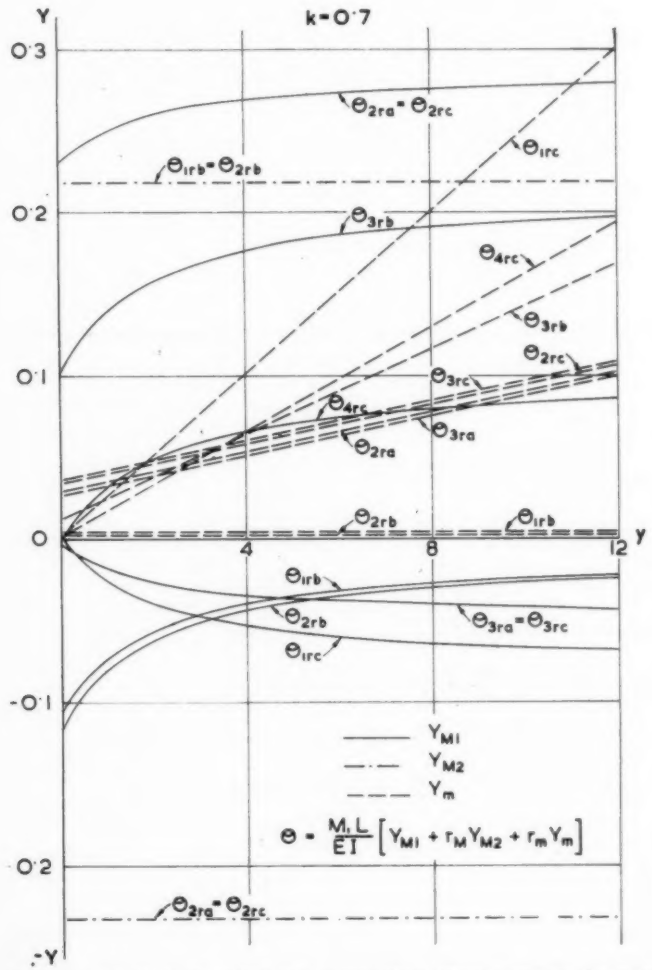


Fig. 4.—Curves relating Y and y for computing Plastic-Hinge Rotations with the use of Equation (5).

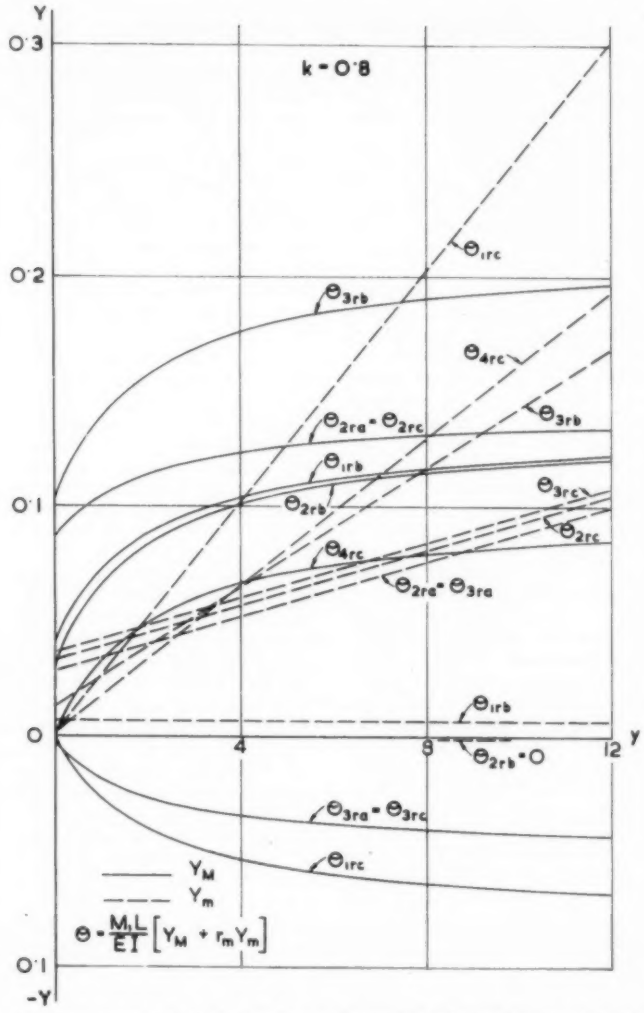


Fig. 5.—Curves relating *Y* and *y* for computing Plastic-Hinge Rotations with the use of Equation (8).

Reference.

(1) A. L. L. Baker: "The Ultimate Load Theory Applied to the Design of Reinforced and Prestressed Concrete Frames." (Concrete Publications, Ltd., London, 1956.) "Recent Research in Reinforced Concrete, and its Application to Ultimate Load Design." (Structural Paper No. 26, Inst. Civ. Engrs., 1951.) "Further Research in Reinforced Concrete, and its Application to Ultimate Load Design." (Paper No. 5894, Inst. Civ. Engrs., 1953.)

Unnecessary Accuracy.

MR. A. E. HOLDAWAY, B.Sc., Ph.D., writes as follows.

Sir,—The Editorial Note on " Unnecessary Accuracy " in your December, 1958, number is of particular interest to me as a designer of reinforced concrete with twenty-seven years' practical experience.

Few designers nowadays appear to be able to detach themselves temporarily from the design problems with which they are daily confronted in order to seek a just appraisal of where modern tendencies in design are leading, or why (as you pointed out) the output per man is very much lower to-day than it was twenty years ago, or why employers discourage unnecessary precision in drawing-office work. The reasons are, however, not difficult to see.

When the writer commenced design work some twenty-seven years ago the output per man per day—in the case of draughtsmen as well as designers—was very appreciably higher than at present. The chief reason was that meticulous accuracy of calculations and the appearance of drawings was then of secondary importance to practical utility. Drawings were then generally done in pencil, neatly and tidily but without the absurdly high finish (in ink) now demanded by some employers. Also, the amount of writing-up on drawings is carried to ridiculous extremes; bar numbers are repeated in a multitude of views, unnecessary sections are drawn, and beams are all drawn in full detail. By adopting the methods suggested by the American Concrete Institute, drawing-office work can be reduced by as much as 80 per cent. compared with some other methods, as the writer has proved in his own work. A further waste of time is the slavish affection for feet and inches, the metric system being far quicker and decidedly more practical.

Modern tendencies are consistently towards further complication of simple processes, so that work that should be quite simple becomes unnecessarily involved. Consider, for example, the time spent in determining bending moments on columns by the methods advocated in the B.S. Code of Practice and in by-laws. The application of this method to a large multiple-story building is extremely laborious and time-consuming, and when the work is done, and the results compared with what a designer—basing his judgment on practical experience—would have calculated as necessary, it is invariably found that the B.S. method wastes steel and, in the writer's opinion, is no nearer the truth. The writer has designed scores of structures quickly and economically without working out the complicated K values and performing the single moment distributions, and not one of these buildings has given the slightest trouble. A fortnight can be spent on the calculations for one shell roof. Most frames can be quickly analysed by short-cut methods from first principles, although some must be analysed completely. Academic accuracy is not necessary in the design of continuous beams where only essentials are required.

Meticulous accuracy is fundamentally illusory since, as Mr. G. P. Manning and others have pointed out, concrete is not an elastic or a homogeneous material, and the theory of bending, which (apart from the load-factor method) forms the basis of design even to-day, is not applicable. As Mr. Manning and others have also stated, there is no exact theory of concrete as such, and unnecessary efforts to achieve " 100 per cent. accuracy " are a waste of time and money.

Twenty-five years ago designers occupying senior posts were far older and more experienced than the younger men who often occupy such posts to-day and who, because they have not the requisite background of practical experience, must necessarily have recourse to academic methods even if they do increase the time taken in designing.

Technical calculations should be subservient to engineering skill. The engineer should first consider the practical problems he has to face, and technical calculations—done quickly and without fictitious " accuracy "—should be merely a means to an end. The policy of " not seeing the wood for the trees " can lead to failures such as those recorded in your Editorial Note.

High-Strength Notched Reinforcement.

We have received the following from Mr. A. Zaslavsky, of the Israel Institute of Technology, Haifa.

Detailed data on the different types of high-strength steels suitable for reinforcement are generally available (see this journal for June 1957), but information about Russian steels is limited. Of special interest is the Avakov cold-notched steel, having an actual proof stress of about 25,000 lb. per square inch. The design proof stress specified by the Russian Codes of Practice of 1955 is not identical with the actual proof stress, as a part of the safety factor—the "homogeneity factor"—is included in the design strength of the material. In addition, the design proof stress is multiplied by a "service factor" which depends on the grade of the steel and the type of stress to which it is subjected. The other two components of the safety factor are a "service factor" for the whole structure and a variable "overload factor" which is assigned to the dead and live loads.

Although its proof stress is not high, the steel has advantages which the writer has also noted in tests with cold-worked steel produced by a similar method. The Avakov steel is a cold-worked steel produced from ordinary round mild-steel bars by alternate notching in two mutually-perpendicular directions (*Fig.* 1). This is effected by a single rolling process, which compresses the diameter to 75 per cent. of its original value and causes some elongation which the writer's tests indicate to be about 8 per cent. (The proof stress mentioned in the foregoing is referred to the original section of the bar.) Some strain-hardening takes place in the narrow intervals between the notches, but most of the original ductility is retained at these sections. The critical strain is about 6 per cent.

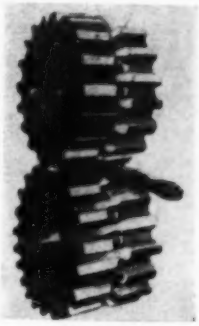


Fig. 3.

The notching produces equal increases of the yield point in tension and compression, and the Russian codes accordingly permit the use of this steel for both purposes. Its surface gives excellent bond and the continuous cold-rolling process is also advantageous in that it permits the steel to be rolled directly from the coils. *Fig.* 2 shows a diagram of the rolling assembly combined with a straightening device (4), a cutter (5), and a receiving arrangement (7). *Fig.* 3 shows a pair of rolls. The average output for diameters of $\frac{1}{2}$ in. and smaller

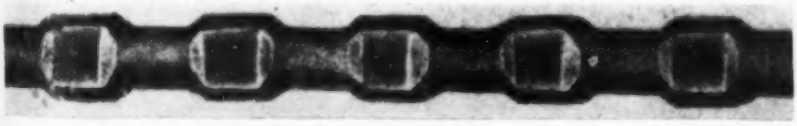


Fig. 1.

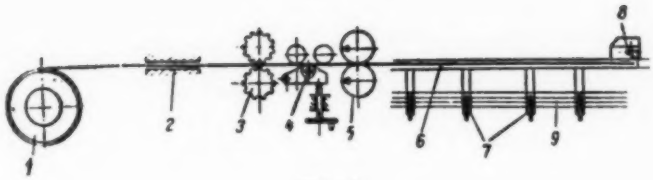


Fig. 2.

is said to be about one ton per hour for an unskilled labourer; the mill is driven by a 10 kW. motor.

Russian tests indicate that the cold-notching of mild steel with a carbon content of 0·28 to 0·37 per cent. raises the yield (proof) stress to about 42,000 lb. per square inch. This steel has also been used with success for prestressed concrete beams of 51 ft. span.

Book Reviews.

"New Ways of Building," Third edition. (London: Architectural Press, Ltd. Price 45*s*.)

THIS work was first published in 1948, and the latest edition has been brought up to date by the inclusion of illustrations of more recent structures. The sections, each written by different authors, comprise concrete, steel, timber, glass, brickwork, light metals and plastics, and insulation. Most of the illustrations will be familiar to readers of architectural and technical periodicals.

"Investigations of Danish Aggregates at the Building Research Station." By F. E. Jones. (Copenhagen: Danish National Institute of Building Research. No price stated.)

THIS is an account of investigations made at the Building Research Station on alkali reactions in concrete. It is stated that the results of this work correspond very well with the results of similar investigations made in Denmark, with the exception of a few deviations which are said to be probably due to a comparison of a Danish flint with English flints which are older and probably have a different structure.

"A Dictionary of Civil Engineering." By John S. Scott. (London: Penguin Books, Ltd. Price 5*s*.)

IT is a pity that the publishers have thought fit to introduce this serious work with a particularly silly "blurb". Within 415 pages definitions are given of some 5000 terms; some of them are not related to civil engineering (for example, "dry cleaning—cleaning of coal by air currents") and could with advantage be omitted in a future edition, but the student of civil engineering may find the work a useful source of reference when he comes across unusual terms.

"Kempe's Engineer's Year Book for 1959." [London: Morgan Bros. (Publishers), Ltd. Two volumes. Price 82*s*. 6*d*.]

THE 1959 edition of this well-known publication is substantially the same as in recent years with much of the subject matter brought up to date. Nearly all branches of engineering are dealt with, but of special interest to civil engineers are the sections in the second volume dealing with surveying, foundations, earthworks, materials, buildings, steelwork, bridges, roads and concrete including reinforced and prestressed concrete, the data in which is in accordance with the latest B.S. Codes. The remaining sections are a valuable source of information on other branches of engineering.

Deutscher Ausschuss für Stahlbeton, Bulletin No. 131: Das Kriechen unbewehrten Betons. By Dr.-Ing. Otto Wagner. (Berlin: W. Ernst & Son. 1958. Price 20 D.M.)

THE results of 186 tests on the creep or flow of plain concrete under sustained load are summarised. The work is intended for the use of research workers. For practical purposes the ultimate creep of gravel concrete is estimated by the method described in an article in this journal for September, 1957. More than 200 references are given, two-thirds of which are to British and U.S. sources.

"Alment om alkali reaktioner i beton." (Alkali Reactions in Concrete.) By P. Nerenst. Printed in the Danish language. (Copenhagen: The Danish National Institute of Building Research. No price stated.)

THIS is an introduction to a series of reports to be issued by the Institute on the effect on concrete of aggregates containing alkalis. It takes the form of a review of the research carried out on this problem in the U.S.A., and gives a brief summary of methods of testing and a bibliography.

Mathematical Methods of Proportioning Aggregates.

MR. K. W. DAY, B.Sc., A.M.I.C.E., A.M.I.E.(Aust.), of Victoria, Australia, writes as follows :

In the early years of this century considerable attention was given to producing a mathematical system of proportioning aggregates based on their fineness modulus. These attempts had only a limited success because the fineness modulus does not represent a fundamental property of the aggregate, namely, its water requirement.

Mr. Tate's system (see this Journal for April 1958) is an elaboration of this work. It is of course possible to " design " a mixture by producing an exact replica of a published mixture found to be successful by a previous user. This, in essence, is the " ideal grading curve " approach ; it normally results in concrete with satisfactory properties but often causes considerable expense due to the necessity of using many separate bins of aggregate and/or hauling special aggregates long distances. Mr. Tate finds it necessary to restrict his method to a standard type of grading and to give different " target " moduli for gradings with different maximum sizes.

For some years now papers have been available drawing attention to the merits of the specific surface as a measure of the suitability of an aggregate. Mr. Stewart[1] pointed out the value of a " surface modulus " based on the specific surface of spherical particles. Mr. Newman and Mr. Teychenné[2] measured the specific surface of a large number of sands and demonstrated that mixtures with equal overall specific surfaces often have the same properties. For the past four years I have used this principle to design concrete mixtures using widely differing gap-graded and continuous aggregates. It has been possible consistently to produce concrete with cube strengths of 6000 lb. per square inch with sands considered unusable by any other system. The D.S.I.R. Road Note No. 4 gives four gradings of mixtures using $\frac{3}{4}$-in. aggregates and four using $1\frac{1}{2}$-in. aggregates which have similar properties. It is interesting to note that the specific surfaces of equivalent $\frac{3}{4}$-in. and $1\frac{1}{2}$-in. gradings are almost identical.

The validity of the specific-surface method is therefore confirmed by theory, experimental verification, a research project which unintentionally reached the same answer, and successful commercial application.

When using the method, the following points should be noted.

(1) Not all mixtures of equal specific surface have the same properties ; an additional requirement is that there must be sufficient sand to fill the voids in the coarse aggregate. If great care is taken in selecting and proportioning two coarse aggregates a sand content of only 15 per cent. can be used, but when 26 per cent. or more of sand can be used then only one single-sized coarse aggregate (whatever its size) is normally required.

(2) The converse that mixtures with different specific surfaces have different properties is universally true.

(3) The difference in specific surface caused by different particle shapes is not reflected to the full extent in their behaviour in concrete, and the writer has found it sufficient to assume that a particular sieve fraction has a constant specific surface.

(4) The true specific surface slightly over-estimates the importance of the lower sieve fractions. The figures given in *Table* 1 have been modified to correct this.

TABLE 1.

Sieve fraction	Modified specific surface (sq. cm. per g.)
$1\frac{1}{2}$ in.–$\frac{3}{4}$ in.	2
$\frac{3}{4}$ in.–$\frac{3}{8}$ in.	4
$\frac{3}{8}$ in.–$\frac{1}{16}$ in.	8
$\frac{1}{16}$ in.–No. 7	16
No. 7–No. 14	27
No. 14–No. 25	39
No. 25–No. 52	58
No. 52–No. 100	81
Passing No. 100	105

(5) The specific surface is used by selecting the curve in Road Note No. 4 which has the desired properties, and

combining the aggregates available so as to give the same specific surface. This is done in the way given by Mr. Tate for the fineness modulus, but will be found to be much more widely applicable (especially with gap gradings).

(6) The writer has not tried this system for hand-placed mixtures with slumps of not more than 1 in., and it may not apply to wet mixtures.

(1) The Design and Placing of High Quality Concrete, by D. Stewart. Spon. 1951.

(2) A Classification of Natural Sands, by A. J. Newman and D. C. Teychenné. Symposium on Mix Design and Quality Control of Concrete. London. 1954.

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The Vacuum Process Applied to Hollow Members.

It is reported that the vacuum process of removing excess water from concrete is extensively used in the U.S.S.R. In

Fig. 1.

the case of hollow members, the present practice is to apply internal suction in conjunction with pressure. The application of this process to a hollow pile is shown in *Fig.* 1.

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Lectures on Building.

THE following lectures have been arranged by the Ministry of Works. Admission is free.

Practical Formwork Design and Construction for Concrete, by J. G. Richardson. N.E. Essex Technical College, Sheepen Road, Colchester. February 23. 7.15 p.m.

Sound Insulation, by J. A. Godfrey. Technical College and School of Art, Collier Road, Cambridge. February 16. 7.30 p.m.

Introduction to Programming and Progressing for Builders, by A. E. Chittenden. Cornwall Technical College, Redruth. February 17. 7 p.m.

Work Study in the Building Industry, by C. A. Francis. Technical College, Denzil Road, London, N.W.10. February 17. 7.15 p.m. And by A. E. Chittenden. Hendford Manor Hall, Hendford, Yeovil. February 18. 7.30 p.m.

Arbitration and Awards, by J. J. Clarke. Technical College, Aylestone Hill, Hereford. February 18. 7.15 p.m.

Safety in the Building Industry, by J. A. Hayward. Schofield Technical College, Park Road, Mexborough. February 19. 7.15 p.m.

Common Defects in Building, by H. J. Eldridge. Technical College, Neath. February 19. 7 p.m.

Corrosion of Metals in Building, by P. Murphy. Institute of Technology, Bradford. February 24. 7.15 p.m.

Dampness in Buildings, by J. P. Latham. College of Further Education, Warwick New Road, Leamington. February 24. 7.15 p.m.

Some Aids to Productivity, by A. E. Chittenden. Cornwall Technical College, Redruth. February 24. 7 p.m.

The R.I.B.A. Form of Contract, by D. Gardam. Harris College, Corporation Street, Preston. February 25. 7.15 p.m.

Prestressing Equipment.

Prestressed Sales, Ltd., of 1 Adelaide Street, London, W.C.2, has been formed as a subsidiary of the Ductube Co., Ltd., to deal with sales oversea of equipment for prestressing concrete in accordance with the Gifford-Udall systems, and to sell in the United Kingdom the new tube anchorage and large strand system devised by Udalls Prestressed Concrete Ltd. The Company will also manufacture Ductex and Ductiflex metal sheathing.

FIFTY YEARS AGO.

From "CONCRETE AND CONSTRUCTIONAL ENGINEERING", January–February, 1909.*

MIXING OF CONCRETE.—In regard to the mixing of concrete, Mr. Marsh lodged a protest against the not unusual practice of defining the proportions as 1 to 4, 1 to 6, etc., which gave no indication of the relative quantities of sand and stone. Such a definition was only permissible when using a ballast or unscreened broken stone, and in such a case it should be clearly specified that a proper quantity of sand should always be maintained in the aggregate, either by the addition of further sand or the screening out of a surplus. Definite proportions of cement, sand, and stone should always be adopted for the best classes of concrete. It was probable that before long the method of proportioning by volumes of cement, sand, and stone would be superseded by the proportioning by weight of cement to volumes of sand and stone. It was impossible always to obtain the same amount of cement in gauge boxes, as the method of filling had a considerable effect on the quantity. The contractor would have a much better control of the amount of cement used when it was proportioned by weight, while the engineer or architect was sure of obtaining the proper quantity.—(From a report of a discussion at the Concrete Institute.)

* "Concrete and Constructional Engineering" appeared in alternate months until September, 1909.

Forming Recesses in Concrete.

A DEVICE for forming a recess in concrete to accommodate an anchor for brickwork is shown in *Fig.* 1. It consists of a piece of rubber, of trapezoidal cross section, in which is embedded a $\frac{5}{16}$-in. Whitworth nut. A hole is drilled in the shuttering, to which the rubber is attached by means of a setscrew which passes through the

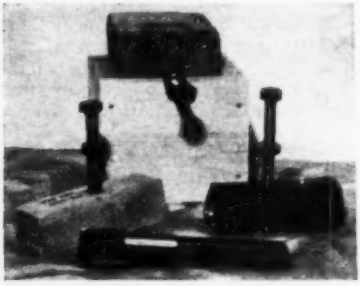

Fig. 1.

hole. When the concrete has hardened the rubber may be removed with the shuttering. Alternatively the bolts may be extracted and the shuttering stripped without disturbing the rubber, which can be pulled out later. The device is supplied by the Alfex Co., of London, who also make an anchor (shown in the illustration) for use with the insert. Other types of anchor can also be used with this system.

Competition for an Elevated Prestressed Concrete Road.

THE Prestressed Concrete Development Group is promoting a competition for the design of an elevated road constructed mainly or entirely in prestressed concrete. Designs must be submitted by June 30 next and questions relating to the conditions must be sent by February 28. It is to be assumed that the road is to be over an existing dual carriageway. The design is to include various types of junctions to and from the main elevated road. The promoters suggest that collaboration between an engineer and an architect is desirable. Full particulars can be obtained from the Prestressed Concrete Development Group, 52 Grosvenor Gardens, London, S.W.1.

DATA FOR PRICING REINFORCED CONCRETE.

Materials.

(Delivered in London area.)

AGGREGATES (per cu. yd.).—Washed sand, 27*s.* 7*d.* Clean shingle : $\frac{3}{4}$ in., 23*s.* 11*d.*; $\frac{1}{2}$ in., 26*s.* 6*d.* Pit ballast, 27*s.*

CEMENT (per ton, delivered at Charing Cross).—Portland cement, 6 tons and upwards, 112*s.* 1 ton to 6 tons, 124*s.*

Rapid-hardening Portland, 10*s.* 6*d.* above ordinary Portland.

Aquacrete and 417, 32*s.* 6*d.* above ordinary Portland.

Colorcrete (buff, red, and khaki), 60*s.* above ordinary Portland.

Snowcrete, 277*s.* 6*d.*

"Super-Cement," 32*s.* 6*d.* per ton above ordinary Portland cement.

High-alumina cement : per ton, 329*s.*; 2 tons, at 318*s.* 9*d.*; 4 tons, at 313*s.* 9*d.*; 6 tons, at 310*s.* 3*d.*

Snowcem paint, 71*s.* per cwt.

SHUTTERING.—1" and 1$\frac{1}{4}$" deal sawn shuttering boards, £95–£100 per standard.

REINFORCEMENT.—Mild steel bars, B.S. 785 (per cwt.) ; $\frac{5}{8}$ in. to 2$\frac{1}{2}$ in., 45*s.* 9*d.*; $\frac{5}{16}$ in. to $\frac{1}{2}$ in., 47*s.* 6*d.*; $\frac{3}{8}$ in., 48*s.* 3*d.*; $\frac{1}{4}$ in., 49*s.* 9*d.*

Materials and Labour.

(Contracts up to £5000. Including 10 per cent profit.)

PORTLAND CEMENT CONCRETE, 1 : 2 : 4.—Foundations, 2*s.* 10*d.* per cu. ft. Columns, 3*s.* 10*d.* per cu. ft. Beams, 3*s.* 7*d.* per cu. ft. Floor slabs, 4 in. thick, 10*s.* 2*d.* per sq. yd.; Do., 5 in., 12*s.* 2$\frac{1}{2}$*d.*; Do., 6 in., 14*s.* 8*d.*; Do., 7 in., 17*s.* 3*d.* Walls 6 in. thick, 16*s.* 3*d.* per sq. yd. Add for hoisting above ground level, 3*s.* 10$\frac{1}{2}$*d.* per cu. yd. Add for rapid-hardening Portland cement, 3*s.* 10$\frac{1}{2}$*d.* per cu. yd.

REINFORCEMENT.—Mild steel bars (B.S. 785), including cutting, bending, fixing, and wire (per cwt.)—$\frac{1}{4}$ in. to $\frac{3}{8}$ in., 90*s.* $\frac{5}{16}$ in. to $\frac{1}{2}$ in., 80*s.* 10*d.* $\frac{5}{8}$ in. to 2$\frac{1}{2}$ in., 72*s.* 8*d.*

SHUTTERING AND SUPPORTS.—Walls, 224*s.* 6*d.* per square.

Floors (average 10 ft. high), 208*s.* 6*d.* per square. In small quantities, 2*s.* 9*d.* per sq. ft.

Columns, average 18 in. by 18 in. (per sq. ft.), 3*s.* 1$\frac{1}{2}$*d.*; in narrow widths, 4*s.* 0$\frac{1}{2}$*d.*

Beam sides and soffits, average 9 in. by 12 in. (per sq. ft.), 3*s.* 2*d.*; in narrow widths, 3*s.* 8*d.*

Raking, cutting, and waste, 7*d.* per lin. ft.

Labour on splays, 3$\frac{1}{2}$*d.* per lin. ft.

Small fillets to form chamfers, 7*d.* per lin. ft.

Wages.

The rate of wages on which the above prices are based are : Carpenters and joiners, 4*s.* 6*d.* per hour (carpenters 2*d.* a day tool money); Labourers, 4*s.* 3$\frac{1}{2}$*d.*; Men on mixers and hoists, 4*s.* 8$\frac{1}{2}$*d.*; Bar-benders, 4*s.* 8*d.*

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